STRUCTURAL ANALYSIS OF AN INDUSTRIAL BUILDING

IN THIS DOCUMENT WE CAN FIND THE PROCEDURE DEVELOPED TO DETERMINE AND CHECK THE STRUCTURAL ELEMENTS OF AN INDUSTRIAL BUILDING MADE OF CONCRETE AND STEEL

FINAL PROJECT

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1 INTRODUCTION

1.1 Introduction to the idea

The first thing that encouraged me to do this project was the area of study. When I had to decide the specialization in Spain, I chose Mechanics of Machines and Structures (Especialidad Mecánica de Máquinas y Estructuras).
The part I liked most of these two was Machines part. After a few years and quite a lot of subjects I have realized that I prefer to focus my interest in the mechanical part of the structures.

The second point of interest of this project is the future one. My idea is to apply for the company “Técnicas Reunidas”. It is a company focussed on the calculation and development of mainly industrial buildings, among other things. I like the idea of this company, and its offer around the world.

So, after knowing this and the area of work, I wanted to know a little more about what is to calculate and design this kind of structure, to have a non-professional idea. It has been quite useful for me.

1.2 Preliminary idea of an industrial building

An industrial building is a building designed for the storage and/or production of any kind of goods, and has to be able to shelter this material, people working inside it and the machines that will be used for this labours.

The size and the equipment needed in each of them will depend on the final objective of the building, the economy of the owner, or the desired amount of product or labour developed inside.

There are some possible materials when designing such a building. The three more extended in Spain are:

- Concrete structure: Is the most common nowadays. It offers a lot of possibilities, as it can be built in pre-manufactured concrete or fresh concrete. This is the most economical option. However, for buildings made of fresh concrete, time of construction is usually larger, as it has to dry before performing the following steps.
- Steel structure: It is growing in popularity. It is easier to construct and has shorter times. The main problem of it is its price, quite more expensive than concrete.
- Mix structure: Can have the best thing of both types. Usually have a steel roof and enclosures, and concrete columns and foundations. With it we can have a better control of the budget and we can save time in the construction.

2 BUILDING

2.1 Scope of the project

The main scope of this project is to design the elements and check the actions that will be applied on the industrial building.

It will be focussed on the calculations, and not in the technological part. So, the manufacturing part won´t be performed. I will perform and calculate the structural response of the building, if it can stand the applied loads and solicitations.

2.2 Initial data of the building

2.2.1 Material

First of all I have to define the material of construction. I will construct a mix structure, as it is the one which has the best relation between quality, price and time of construction.

The roof will be made of steel sheets standing on the purlins, which will be made of steel as well. These purlins will be fixed to a delta or graded beam. The delta beam will be made of pre-manufactured reinforced concrete, in order to save time.

Columns will be made of pre-manufactured (pre-cast) reinforced concrete, and its technical data will be given in their technical leaflet. A concrete beam-channel will be placed on top of them, in order to give rigidity to the structure and to be able to place a channel for the evacuation of water from the roof . It will be made of concrete.

Finally, the footings will be performed in concrete, and I will have attachment beams between them in order to give more stability to the building and to support the panels for the enclosures, made of concrete as well. The entrance for the building won´t affect anything but the weight for the attachment beam, and it will be less than the enclosures, so it will stand it.

2.2.2 Dimensions and shape

The building will be $15\ metres$ wide, $40.4\ metres$ long and $8.2\ metres$ high in the top part of the roof. This means that dimensions will be $15x40.4x8.2\ m$. It has a rectangular top view. There will be a frame every $8\ m$, which is a total of 6 frames.

The roof will be pitched or gable roof. It will be symmetric by both sides, and the slope of each side will be 10%. Columns will be $7\ m$ high from the terrain, but other extra $0.8\ m$ will be submerged into the calyx of the footings. So it totally makes $7.8\ m$.

Footings will be designed and checked after actions have been calculated.

3 TERRAIN REQUIREMENTS

The building will be built in an industrial area in Madrid. The first thing that I have to do is to define the kind of soil or terrain, and classify the building according to it. This definition should be done by performing some essays in the terrain.

Definition of the construction:

According to table 3.1 of the Basic Document SE_C:

Tabla 3.1. Tipo de construcción

Tipo	Descripción (1)
C-0	Construcciones de menos de 4 plantas y superficie construida inferior a 300 m^2
$\boxed{\text{C-1}}$	$\boxed{\text{Otras construcciones de menos de 4 plantas}}$
C-2	Construcciones entre 4 y 10 plantas
C-3	Construcciones entre 11 a 20 plantas
C-4	Conjuntos monumentales o singulares, o de más de 20 plantas.

(1) En el cómputo de plantas se incluyen los sótanos.

The building is classified as type C-1, because it will have less than 4 floors, and the area occupied will be $40.4 \cdot 15 = 606\ m^2 > 300\ m^2$.

Definition of the terrain:

According to table 3.2 of the Basic Document SE-C:

Tabla 3.2. Grupo de terreno

Grupo	Descripción
T-1	Terrenos favorables: aquellos con poca variabilidad, y en los que la práctica habitual en la zona es de cimentación directa mediante elementos aislados.
$\boxed{\text{T-2}}$	Terrenos intermedios: los que presentan variabilidad, o que en la zona no siempre se recurre a la misma solución de cimentación, o en los que se puede suponer que tienen rellenos antrópicos de cierta relevancia, aunque probablemente no superen los 3,0 m.

The terrain will be of type T-2. This means that the terrain is not perfect. It has good conditions but can be less uniform as expected. It is the most common in this areas.

The admissible stress for the terrain will be $\sigma_{terrain} = 2.20\ kg/cm^2$, and the type of foundation appropriate for it is isolated rigid footings, which will have attachment beams between them to give better structural response and, mainly, to support the panels of the enclosures.

This terrain will be in good superficial conditions for the construction: Flat, without levelling operations needed. It will just need the holes for the footings and the foundation beams.

4 VARIABLE LOADS IN ROOF

4.1 Snow load

The distribution and intensity of the snow load on a building, or in particular on a roof, depends on the local climate, the type of precipitation, relief of environment, the shape of the building or deck, the effects of wind, and the heat exchange in the external parameters. When these roofs are accessible to people or vehicles, we have to consider the possible accumulations due to artificial redistribution of snow.

4.1.1 Determination of snow load “q_n”

According to Basic Document SE-AE for actions in construction, the snow load per surface unit q_n can be obtained by the formula:

$$q_n = \mu \cdot s_k$$

μ is the coefficient of shape of the roof

s_k is the characteristic value of the snow load

4.1.2 Characteristic value of snow load “s_k”

The characteristic value of snow load can be obtained from table 3.8 of the Basic Document SE-AE.

Capital	Altitud m	s_k kN/m^2
Guadalajara	680	0,6
Huelva	0	0,2
Huesca	470	0,7
Jaén	570	0,4
León	820	1,2
Lérida / *Lleida*	150	0,5
Logroño	380	0,6
Lugo	470	0,7
Madrid	660	0,6
Málaga	0	0,2
	40	

In this case, the building is placed in Madrid, which is at 660m high, so the value will be $s_k = 0.6\ kN/m^2$.

4.1.3 Coefficient of shape of the roof "μ"

The wind may accompany or follow the snow, resulting in an irregular deposit of snow on the roofs. Therefore, the thickness of the snow cover can be different in each side of the roof.

In a pitch in which there is no impediment for the sliding of snow, shape coefficient has different value according to the slope of the roof:

1. Roofs for less than or equal to 30 º inclination is equal to 1

2. For roofs with inclination greater than or equal to 60 ° is equal to 0

3. Roofs with intermediate slope values are interpolated linearly.

In my case, the cover of the warehouse to perform has a slope of 10%, and assuming that there is no impediment to slipping of the snow, the coefficient will be:

$arctg\alpha = \frac{\%roof}{100} = \frac{10}{100} = 0.11 \rightarrow \alpha = 5.7^{\circ} \leq 30^{\circ}$

So, this is case 2, and I have $\mu = 1$.

4.1.4 Final value of "q_n"

Introducing the coefficients obtained above, the result is:

$$q_n = \mu \cdot s_k = 1 \cdot 0.6 = 0.6\frac{kN}{m^2} = 60\frac{kg}{m^2}$$

4.2 Wind loads

The distribution and the value of the pressures exerted by the wind on a building and the resulting forces depend on the shape and dimensions of the construction, the nature and permeability of the surface and the direction and intensity of the wind.

4.2.1 Determination of the action of the wind on the roof "q_e"

We call static action or pressure of the wind, q_e, to the force perpendicular to the surface of each exposed wall of the building. It can be defined with the next formula:

$$q_e = q_b \cdot c_e \cdot c_p$$

Where:

q_b is the dynamic pressure of the wind
c_e is the exposure coefficient
c_p is the Eolic or pressure coefficient

I have to take into account that the static pressure of the wind depends on the height where we are calculating. So, I will calculate it in the most unfavourable point, which is the top of the roof.

4.2.2 Determination of dynamic pressure of the wind "q_b"

As a rough value, for all the Spanish territory, we can estimate this value as q_b =0.5kN/m^2.

As a more precise value, we can obtain it with the following formula:

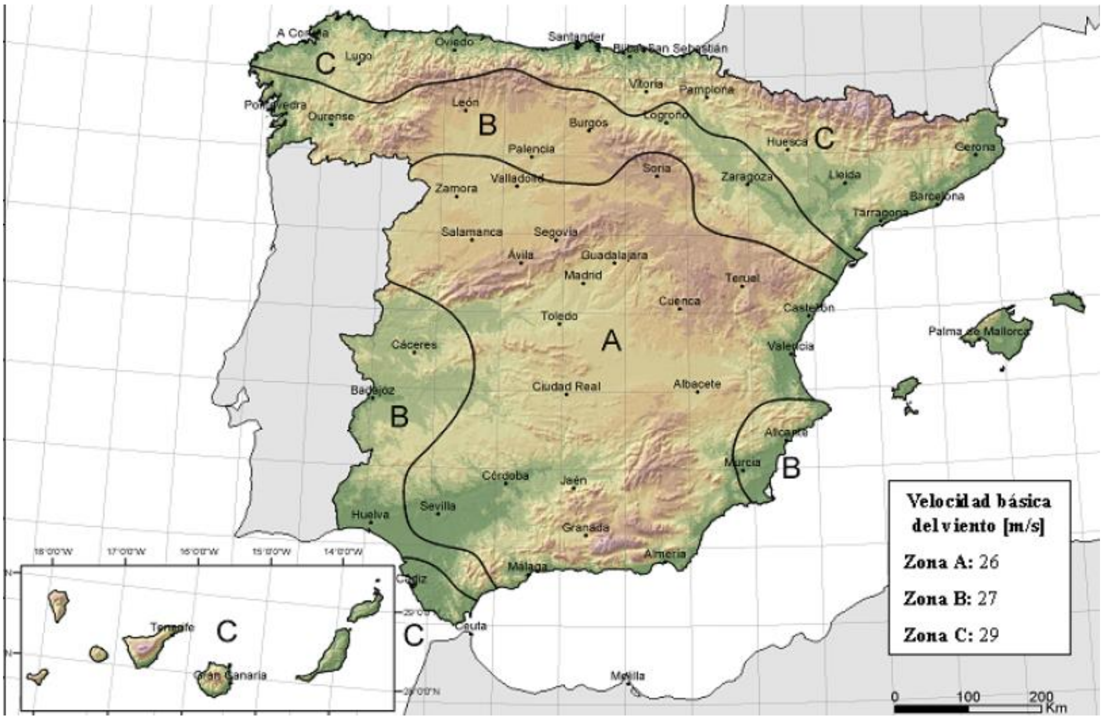
$$q_b = 0.5 \cdot \delta \cdot v_b^2$$

δ is the density of the air

v_b is the basic value of the wind speed

The density of the air depends on the height or temperature, among other values. It can be taken as $1.25\ kg/m^2$.

For the basic value of the wind speed we have table D.1 of the Basic Document SE-AE:



Its value for our case, case A, is $26\ m/s$.

So,

$$q_b = 0.5 \cdot \delta \cdot v_b^2 = 0.5 \cdot 1.25 \cdot 26^2 = 422.5 \frac{kg}{m \cdot s^2} = 0.42 \frac{kN}{m^2}$$

I will use q_b =0.5kN/m^2, as it gives a better security coefficient for the calculations.

4.2.3 Exposure coefficient "c_e"

The exposure coefficient takes into account the effects of turbulence caused by the relief and topography. It will vary depending on the height of the point considered and the degree of roughness of the environment where it is located.

Its value can be obtained from the Basic Document SE-AE, taking into account two things:
1. Height of the top part. In this case it will be $7.0\ m$ of the walls + $1.2\ m$ of frame $= 8.2\ m$

2. Location of the building, which will be an Industrial area. This kind of area is type IV.

With this two values we go to the table 3.4 from Basic Document SE-AE:

Tabla 3.4. Valores del coeficiente de exposición c_e

	Grado de aspereza del entorno	Altura del punto considerado (m)							
		3	6	9	12	15	18	24	30
I	Borde del mar o de un lago, con una superficie de agua en la dirección del viento de al menos 5 km de longitud	2,4	2,7	3,0	3,1	3,3	3,4	3,5	3,7
II	Terreno rural llano sin obstáculos ni arbolado de importancia	2,1	2,5	2,7	2,9	3,0	3,1	3,3	3,5
III	Zona rural accidentada o llana con algunos obstáculos aislados, como árboles o construcciones pequeñas	1,6	2,0	2,3	2,5	2,6	2,7	2,9	3,1
IV	Zona urbana en general, industrial o forestal	1,3	1,4	1,7	1,9	2,1	2,2	2,4	2,6
V	Centro de negocio de grandes ciudades, con profusión de edificios en altura	1,2	1,2	1,2	1,4	1,5	1,6	1,9	2,0

As we don't have exact value for it, I must interpolate:

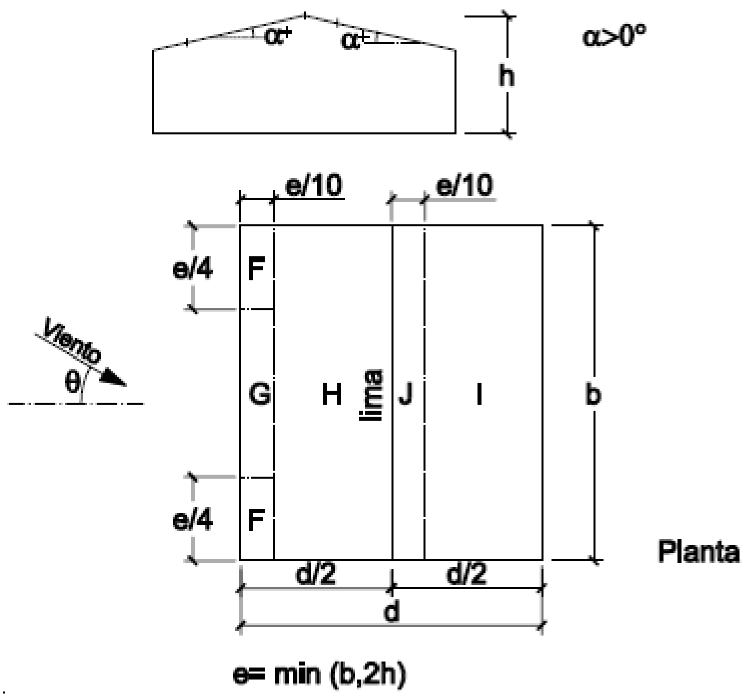
$$\frac{1.7-1.4}{9-6} = \frac{1.7-c_e}{9-8.2} \rightarrow c_e = 1.62.$$

4.2.4 Eolic or pressure coefficient "c_p"

The pressure coefficient depends on the relative direction of the wind, shape of the roof, the position of the element considered and its area of influence.

Its value is set, for various simple forms of housing construction, in the tables of the Basic Document SE-AE, where they give values obtained as the worst ones of the wind directions. A positive value indicates pressure, and a negative refers to the suction.

For the calculations I will take into account the highest value for pressure, because for suction the limit will be determined by the strength of the screws fixing the roof. It is a pitched roof, so from table D.6:



It depends on the area of each zone.
Dimensions of the building are $b = 40.4\ m$, $h = 8.2\ m$, $d = 15\ m$.

Knowing that $e = min(b, 2h) = min\ (40.4, 15.5) = 16.4\ m$.

Distinguishing between $A < 10\ m^2$ and $A \geq 10\ m^2$.

$F = 6.72\ m^2 < 10\ m^2$;$G = 52.81\ m^2 > 10\ m^2$; $H = 236.74\ m^2 > 10\ m^2$;
$I = 236.74\ m^2 > 10\ m^2$; $J = 66.26\ m^2 > 10\ m^2$

Pendiente de la cubierta α	A (m²)	Zona (según figura)				
		F	G	H	I	J
5º	≥ 10	-1,7 +0,0	-1,2 +0,0	-0,6 +0,0	-0,6	0,2 -0,6
	≤ 1	-2,5 +0,0	-2 +0,0	-1,2 +0,0	-0,6	0,2 -0,6
15º	≥ 10	-0,9 0,2	-0,8 0,2	-0,3 0,2	-0,4 +0,0	-1 +0,0
	≤ 1	-2 0,2	-1,5 0,2	-0,3 0,2	-0,4 +0,0	-1,5 +0,0

According to this, the values are the ones of zone J, which is the top zone. As I said, I take the biggest positive values for pressure. In $\alpha = 5^\circ$ we have 0.2 and in $\alpha = 15^\circ$ we have 0.0.

My case is $\alpha = 5.7^\circ$, so I have to interpolate:

$$\frac{15-5}{0-0.2} = \frac{15-5.7}{0-c_p} \rightarrow 10 \cdot c_p = 1.86 \rightarrow c_p = 0.186$$

4.2.5 Final value of "q_e"

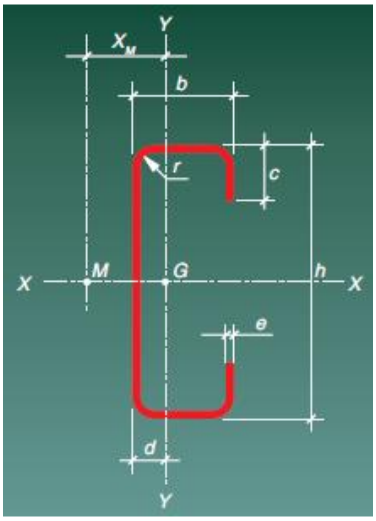
Introducing all the parameters obtained above, the result is:

$$q_e = q_b \cdot c_e \cdot c_p = 0.5 \cdot 1.62 \cdot 0.186 = 0.151\ \frac{kN}{m^2} = 15.1\ \frac{kg}{m^2}$$

5 ROOF PURLINS

5.1 Type of purlin

The purlins will be steel ones. The technical data for this purlins is:



MODELO	Dimensiones (mm) h	b	c	r	espesor	AREA (cm²)	PESO (kg/m)	PERIMETRO (m²/m)	d (cm)
CM-301	80	40	15	2.5	2.0	3.52	2.76	0.356	1.46
CM-302	80	40	15	2.5	2.5	4.34	3.41	0.352	1.46
CM-304	100	40	15	2.5	2.0	3.92	3.08	0.396	1.32

Other values of interest:

$M_f = 20.50\ kN \cdot m\ ;\quad M_u = 31.20\ kN \cdot m;\quad V_u = 25.60\ kN$

5.2 Type of steel sheets for the roof

Tabla 6 – Información técnica de chapa de acero BC18		
Espesor (mm)	0.40	0.45
Peso (Kg/m2 útil)	3.85	4.33
Peso (Kg/m lineal)	3.80	4.28
Momento de inercia (cm4/m útil)	1.84	2.07
Módulo resistente (cm3/m útil)	2.04	2.30

5.3 Determination of limit states

In order to verify the limit states for the determination of the effects of the actions in the purlins, I will take into account the wind load, the snow load, the self-weight of the roof and the self-weight of the purlins.

I will consider that the behaviour will be acceptable if, when dimensioning, the effect of the loads doesn´t overcome the limit value admissible for that effect. This means that an structural element has enough strength if:

$$E_d \leqslant R_d$$

E_d is the value of the effect of actions.

R_d is the value calculated for the strength of the structural element.

The values that have to be calculated are the ones specified in the technical leaflet of the purlins. In this case I have:

$M_f = 20.50\ kN \cdot m$ → Maximum admissible bending moment without enlargement factor (Service Limit State)

$M_u = 31.20\ kN \cdot m$ → Maximum admissible bending moment with enlargement factor (Ultimate Limit State)

$V_u = 25.60\ kN$ → Maximum admissible shear stress with enlargement factor (Ultimate Limit State)

5.4 Combination of actions

The effects of the actions applied on the purlins are determined by the combination of actions and influences simultaneously.

As these actions can produce irreversible effects, they must be determined depending on type.

- For Serviceability Limit State (SLS) we have:

$$\sum_{j\geq 1} G_{k,j} + P + Q_{k,1} + \sum_{i>1} \Psi_{0,i} \cdot Q_{k,i}$$

Here, we consider the simultaneous action of:

1. All permanent actions
2. Each single variable action
3. The other variable actions, taking into account the simultaneous combination between them, with the coefficient Ψ_0.

- For Ultimate Limit State (ULS) we have:

$$\sum_{j\geq 1} \gamma_{G,j} \cdot G_{k,j} + \gamma_P \cdot P + \gamma_{Q,1} \cdot Q_{k,1} + \sum_{i>1} \gamma_{Q,i} \cdot \Psi_{0,i} \cdot Q_{k,i}$$

Here we consider the partial security coefficient, γ_i.

According to this, we have to perform different hypothesis for the variables acting on the purlins, fitting that:

$M_f = 20.50\ kN \cdot m \geq \frac{l^2 \cdot \sum q_i}{8}$ → Bending moment for the cracking of the purlins

$M_u = 31.20\ kN \cdot m \geq \frac{l^2 \cdot \sum q_i \cdot \gamma_i}{8}$ → Ultimate bending moment stood by the purlins

$V_u = 25.60\ kN \geq \frac{l \cdot \sum q_i \cdot \gamma_i}{2}$ → Ultimate shear stress stood by the purlins

Where:

q_i Add of the loads to take into account, having in consideration the distance between the purlins, "s_i", being these loads the following ones:

$$q_i = q_{self-weight\ purlin} + q_{sheet} \cdot s_i + q_n \cdot s_i + q_e \cdot s_i$$

$q_{self-weight\ purlin} = 0.0308 \frac{kN}{m}$ given data

$q_{sheet} = 0.0428 \frac{kN}{m}$ given data

$q_n = 0.6 \frac{kN}{m}$ previously obtained

$q_e = 0.151 \frac{kN}{m}$ previously obtained

l is the length of the purlin, taking into account support points

$$l = l_{total} - \frac{l_{support\ 1} + l_{support\ 2}}{2}$$

Where:

l_{total} is the total length of the purlin, which is $l_{total} = 8\ m$ in this case

$l_{support\ 1}$ is the length of the first support

$l_{support\ 2}$ is the length of the second support

For this case, the length of both supports will be 0.1 metres.

So, as a final result we obtain: $l = l_{total} - \frac{l_{support\ 1}+l_{support\ 2}}{2} = 8 - 0.1 = 7.9\ m$

5.5 Verification of the Serviceability Limit State (SLS)

As I have previously said, I have to check that for every single variable acting on the purlins, the effect of them doesn´t reach the limit value accepted.

$M_f = 20.50\ kN \cdot m \geq \frac{l^2 \cdot \Sigma q_i}{8} =$, taking one action each time without Ψ_0.

Applying this formula for each different variable action, and taking into account that the loads depend on the distance between purlins, "$s_i{}'$", we obtain the following hypothesis:

- Hypothesis first (Snow)

$$20.50\ kN \cdot m = \frac{l^2 \cdot (q_{self-weight\ purlin} + q_{sheet} \cdot s_1{}' + q_n \cdot s_1{}' + q_e \cdot s_1{}' \cdot \Psi_e)}{8}$$

- Hypothesis second (Wind)

$$20.50\ kN \cdot m = \frac{l^2 \cdot (q_{self-weight\ purlin} + q_{sheet} \cdot s_2{}' + q_n \cdot s_2{}' \cdot \Psi_n + q_e \cdot s_2{}')}{8}$$

Applying the simultaneity coefficients of table 4.2 from the Basic Document SE, we get that Ψ is 0.5 for snow (Ψ_n) when height is less than 1000 m and 0.6 for wind (Ψ_e). So:

Coeficientes de simultaneidad ψ	
	ψ_0
Nieve→ Para altitudes ≤ 1000m "ψ_n"	0.5
Viento "ψ_e"	0.6

- Hypothesis first

$$20.50\ kN \cdot m = \frac{7.9^2 \cdot (0.0308 + 0.0428 \cdot s_1{}' + 0.6 \cdot s_1{}' + 0.151 \cdot s_1{}' \cdot 0.6)}{8}$$

$$\rightarrow \quad s_1{}' = 3.54\ m$$

- Hypothesis second

$$20.50\ kN \cdot m = \frac{7.9^2 \cdot (0.0308 + 0.0428 \cdot s_2{}' + 0.6 \cdot s_2{}' \cdot 0.5 + 0.151 \cdot s_2{}')}{8}$$

$$\rightarrow \quad s_2{}' = 5.27\ m$$

As we can see, the most restrictive value is $s_1{}' = 3.54\ m$. So, before calculating the ULS, this will be the maximum value for the distance between purlins.

5.6 Verification of the Ultimate Limit State (ULS)

In order to be sure that the previous distance is correct, I first have to check that the effect of the actions, when applying the enlargement factor "γ_i" doesn´t overcome the stabilised value for the ULS. This is:

$M_u = 31.20\ kN \cdot m \geq \frac{l^2 \cdot \Sigma q_i \cdot \gamma_i}{8}$ and $V_u = 25.60\ kN \geq \frac{l \cdot \Sigma q_i \cdot \gamma_i}{2}$ taking just one different action without Ψ_0.

- Hypothesis first (Snow)

$$31.20\ kN \cdot m = \frac{l^2 \cdot (q_{s-w} \cdot \gamma_{s-w} + q_{sheet} \cdot s_1 \cdot \gamma_{s-w} + q_n \cdot s_1 \cdot \gamma_{var} + q_e \cdot s_1 \cdot \Psi_e \cdot \gamma_{var})}{8}$$

- Hypothesis second (Wind)

$$31.20\ kN \cdot m = \frac{l^2 \cdot (q_{s-w} \cdot \gamma_{s-w} + q_{sheet} \cdot s_2 \cdot \gamma_{s-w} + q_n \cdot s_2 \cdot \Psi_n \cdot \gamma_{var} + q_e \cdot s_2 \cdot \gamma_{var})}{8}$$

- Hypothesis third (Snow)

$$25.60\ kN = \frac{l \cdot (q_{s-w} \cdot \gamma_{s-w} + q_{sheet} \cdot s_3 \cdot \gamma_{s-w} + q_n \cdot s_3 \cdot \gamma_{var} + q_e \cdot s_3 \cdot \Psi_e \cdot \gamma_{var})}{2}$$

- Hypothesis fourth (Wind)

$$25.60\ kN = \frac{l \cdot (q_{s-w} \cdot \gamma_{s-w} + q_{sheet} \cdot s_4 \cdot \gamma_{s-w} + q_n \cdot s_4 \cdot \Psi_n \cdot \gamma_{var} + q_e \cdot s_4 \cdot \gamma_{var})}{2}$$

Applying the simultaneity coefficients used in the SLS (table 4.2 of Basic Document SE) and the partial coefficients of security of table 4.1 of the Basic Document SE, we obtain

Coeficientes parciales de seguridad γ para las acciones		
Tipo de verificación	Tipo de acción	Situación persistente o transitoria desfavorable
Resistencia	Permanente Peso propio, peso del terreno(γ_{peso})	1.35
	Variable (γ_{var})	1.50

- Hypothesis first

$$31.20\ kN \cdot m = \frac{7.9^2 \cdot (0.0308 \cdot 1.35 + 0.0428 \cdot s_1 \cdot 1.35 + 0.6 \cdot s_1 \cdot 1.5 + 0.151 \cdot s_1 \cdot 0.6 \cdot 1.5)}{8} \rightarrow \ s_1 = 3.62\ m$$

- Hypothesis second

$$31.20\ kN \cdot m = \frac{7.9^2 \cdot (0.0308 \cdot 1.35 + 0.0428 \cdot s_1 \cdot 1.35 + 0.6 \cdot s_1 \cdot 0.5 \cdot 1.5 + 0.151 \cdot s_1 \cdot 1.5}{8} \rightarrow \ s_2 = 5.40\ m$$

- Hypothesis third

$$25.60\ kN = \frac{7.9 \cdot (0.0308 \cdot 1.35 + 0.0428 \cdot s_3 \cdot 1.35 + 0.6 \cdot s_3 \cdot 1.5 + 0.151 \cdot s_3 \cdot 0.6 \cdot 1.5)}{2} \rightarrow \ s_3 = 5.89\ m$$

- Hypothesis fourth

$$25.60\ kN = \frac{7.9 \cdot (0.0308 \cdot 1.35 + 0.0428 \cdot s_3 \cdot 1.35 + 0.6 \cdot s_3 \cdot 0.5 \cdot 1.5 + 0.151 \cdot s_3 \cdot 1.5)}{2} \rightarrow \ s_4 = 8.78\ m$$

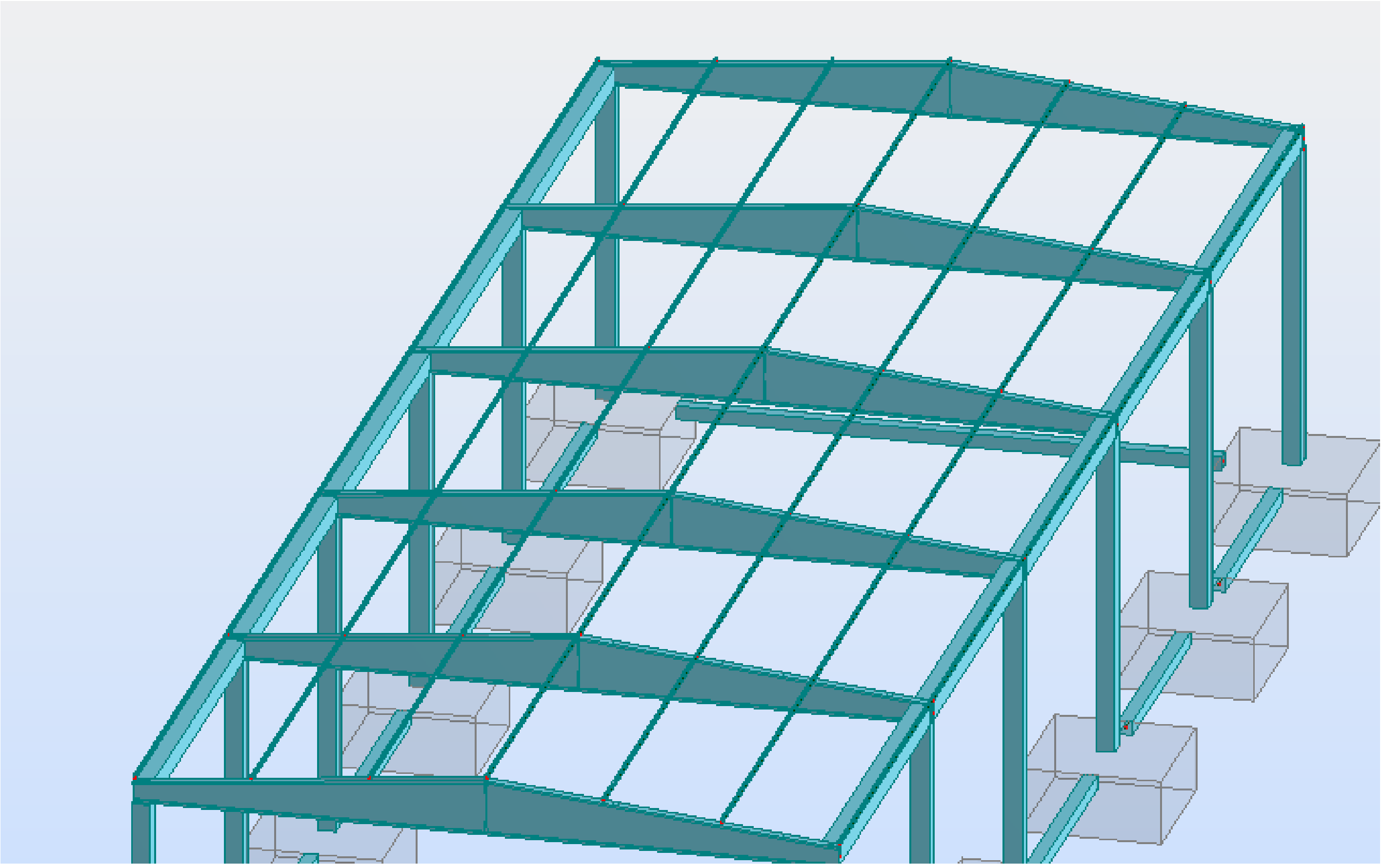
As we can see, we have as restrictive value ${s_1}' = 3.54\ m$. As the distance to cover is:

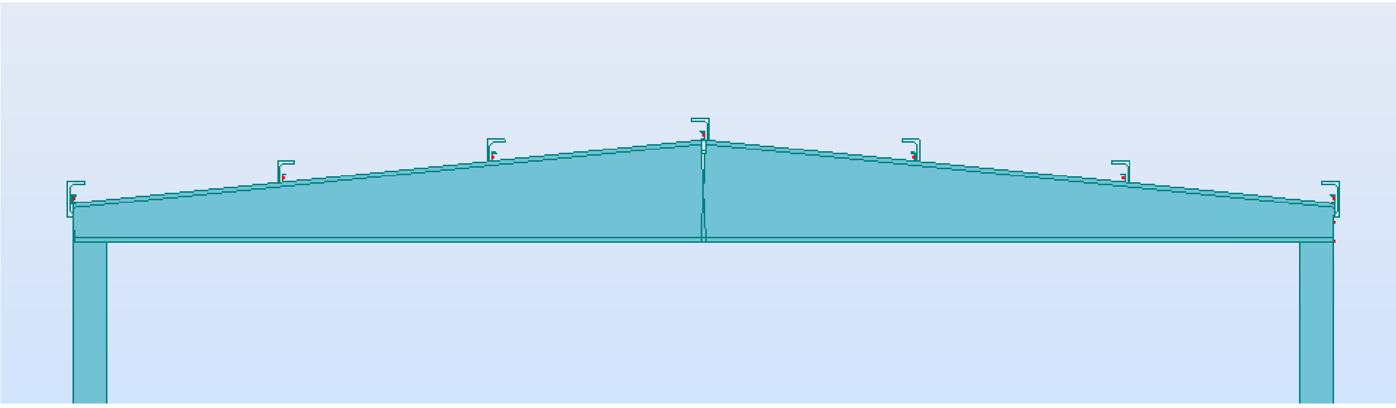
$x^2 = (b/2)^2 + (h_{delta})^2 = 7.5^2 + 0.75^2 = 7.53\ m$, I will use:

$$s_{final} = \frac{7.53}{3} = 2.51\ m$$

So, this value will be the design value for the distance between purlins.

5.7 Final disposition of the purlins





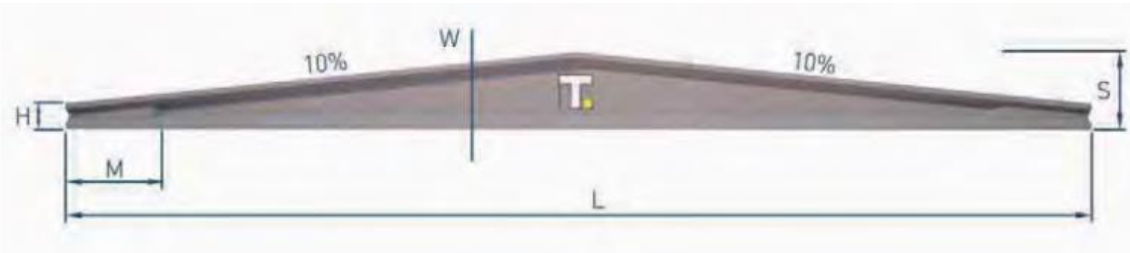
6 CALCULATION OF GRADED OR DELTA BEAMS

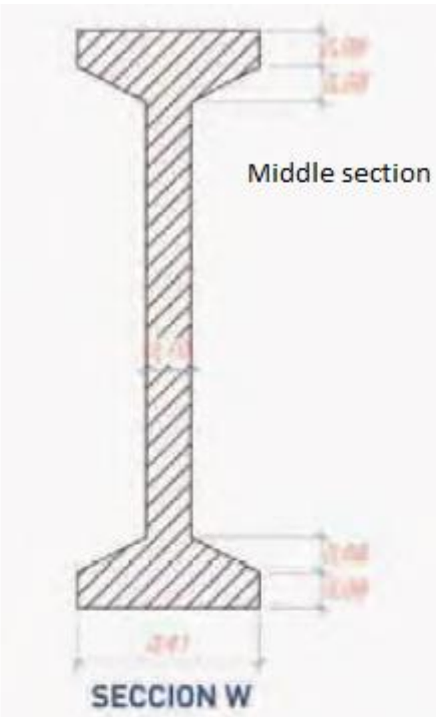
6.1 Graded or delta beam

It is a beam of variable section, the edge part of it will be completely solid, while on the centre part it will be a little lightly. Due to that, it can be considered as an element of constant section, as for practical effects it will be the same.

In order to check the limit states, I will do it only in the section where the bending moment and the shear are maximum.

This is the delta beam I will be using:





L (m)	H (m)	S (m)	M (m)	PESO (T)
10,00	0,50	1,00	0,50	3,561
11,00	0,45		0,10	3,915
12,00	0,40		0,15	4,242
13,00	0,45	1,10	0,10	4,760
14,00	0,40		0,15	5,086
15,00	**0,45**	1,20	**0,10**	**5,649**
16,00	0,40		0,15	[illegible]
17,00	0,45	1,30	0,10	6,584
18,00	0,40		0,15	6,910

Other values of interest:

$M_u = 850\ kN \cdot m$; $V_d = 215\ kN$; $V_{u1} = 710\ kN$; $d = 460\ mm$

$s_t = 100\ mm$; $V_{u2} = 284.70\ kN$; $M_f = 708.20\ kN \cdot m$; $W = 0.2\ m$

6.1.1 Verification of the Ultimate Limit State (ULS)

This limit state is performed for all the situations that can produce a miss of service of the structure or a part of it.

For the beam, I will check the bending moment applied "M_d" and the shear applied "V_d".

To verify this limits, the following condition must be satisfied:

$$S_d \leq R_d$$

Where S_d is the calculated value for the actions and R_d is the value of the structural response.

6.1.1.1 Verification of the applied bending moment "M_d"

The maximum bending moment for the delta beam is $M_u = 850\ kN \cdot m$.

So, I will have to verify that $M_d \leq M_u$, being $M_d = \sum \frac{q_i \cdot l^2}{8} \cdot \gamma_i$, including the enlargement coefficient.

q_i are the loads that must be considered. I mustn´t include the wind load, because for gable roofs the pressure and suction are cancelled between them.

Self-weight: $\frac{P_{s-w}}{l_{total}} = \frac{5649}{15} = 376.6\ \frac{kg}{m} = 3.76\ \frac{kN}{m} = q_{s-w}$

Sheet: $q_{sheet} \cdot l_{intercolumns} = 4.28 \cdot 8 = 34.24 \frac{kg}{m} = 0.3424 \frac{kN}{m} = q'_{sheet}$

Snow: $q_n \cdot l_{intercolumns} = 60 \cdot 8 = 480 \frac{kg}{m} = 4.80 \frac{kN}{m} = q'_n$

l is the length of calculation of the delta beams, taking into account that the supporting head of the beam will be 40 cm for each end:

$$l = l_{total} - \frac{l_{support\ 1} + l_{support\ 2}}{2} = 15 - 0.4 = 14.6\ m$$

γ is the partial security coefficient for actions of table 4.1 of the Basic Document SE, which is
$\gamma_{s-w} = 1.35$ for self-weights and permanent actions.
$\gamma_{s-w} = 1.5$ for variable actions and overloads.

$$M_d = M_{s-w} + M_{sheet} + M_n$$

$$M_{s-w} = \frac{q_{s-w} \cdot l^2}{8} \cdot \gamma_{s-w} = \frac{3.76 \cdot 14.6^2}{8} \cdot 1.35 = 135.47\ kN \cdot m$$

$$M_{sheet} = \frac{q'_{sheet} \cdot l^2}{8} \cdot \gamma_{s-w} = \frac{0.3424 \cdot 14.6^2}{8} \cdot 1.35 = 12.32\ kN \cdot m$$

$$M_n = \frac{q'_n \cdot l^2}{8} \cdot \gamma_{var} = \frac{4.80 \cdot 14.6^2}{8} \cdot 1.5 = 191.84\ kN \cdot m$$

So, $M_d = 135.47 + 12.32 + 191.84 = 339.62\ kN \cdot m \leqslant 850\ kN \cdot m$, and the rule is verified.

6.1.1.2 Verification of the applied shear "V_d"

In order to know if I can consider the transversal reinforcement of the delta beam when calculating the ultimate shear stood, according to article 44.2.3.4.1 from the EHE-08, the next conditions for the maximum distance "s_t" must be satisfied. It will be checked at the end of the beam, as it is the section where shear will be greater:

$s_t \leqslant 0.60 \cdot d \cdot (1 + cotg\alpha)$ and $s_t \leqslant 450\ mm$

If $\frac{1}{5} \cdot V_{ul} < V_d \leqslant \frac{2}{3} \cdot V_{ul}$

where $V_d = 215\ kN$; $V_{u1} = 710\ kN$; $d = 460\ mm$;

$\alpha = 90°$; $s_t = 100\ mm$

$\frac{1}{5} \cdot V_{u1} < V_d \leqslant \frac{2}{3} \cdot V_{u1} \rightarrow 142 < 215 \leqslant 473.3\ kN \rightarrow$ satisfied

$s_t \leqslant 0.60 \cdot d \cdot (1 + cotg\alpha) \rightarrow s_t \leqslant 276\ mm \rightarrow$ satisfied

$s_t \leqslant 450\ mm \rightarrow$ satisfied

Watching this, I can say that it is possible to consider the shear reinforcement, as the distance is lower than admissible.

The data for the beam are:
$V_{u1} = 710\ kN$ → fatigue shear due to compression of the web.

$V_{u2} = 284.70\ kN$ → fatigue shear due to stress of the web.

For an structural element with shear reinforcement I have to verify that:
$V_d \leqslant V_{u1}$ and $V_d \leqslant V_{u2}$

Being V_d the following value:

$$V_d = V_{s-w} + V_{sheet} + V_n$$

$$V_{s-w} = \frac{q_{s-w} \cdot l}{2} \cdot \gamma_{s-w} = \frac{3.76 \cdot 14.6}{2} \cdot 1.35 = 37.11\ kN$$

$$V_{sheet} = \frac{q'_{sheet} \cdot l}{2} \cdot \gamma_{s-w} = \frac{0.3424 \cdot 14.6}{2} \cdot 1.35 = 3.37\ kN$$

$$V_n = \frac{q'_n \cdot l}{2} \cdot \gamma_{var} = \frac{4.80 \cdot 14.9}{2} \cdot 1.5 = 52.56\ kN$$

So, $V_d = 37.11 + 3.37 + 52.56 = 93.05\ kN \leqslant 284.7\ kN \leqslant 710\ kN$ → satisfied.

6.1.2 Verification of the Serviceability Limit State (SLS)

Serviceability Limit States take into account all the situations where the requirements of functionality, commodity or aspect are not satisfied. For the delta beams I will verify:

- Bending moment for cracking "M_f"
- Maximum admissible displacement or strain "Δfl_{total}"

When checking these limits, the next condition must be satisfied:

$$E_d \leq C_d$$

E_d is the value of the effect of the actions

C_d is the value for admissible limit of the section

6.1.2.1 Verification of the bending moment for cracking "M_f"

According to table 8.2.2 of the EHE-08, the elements of concrete in roofs of buildings where the annual average rainfall is less than 600 mm are subjected to a general class of Exposition of type IIb.

Going to table 5.1.1.2 of the EHE-08:

Clase de exposición, según artículo 8°	$w_{máx}$ [mm]	
	Hormigón armado (para la combinación cuasipermanente de acciones)	Hormigón pretensado (para la combinación frecuente de acciones)
I	0,4	0,2
IIa, IIb, H	0,3	0,2[1]

For this element the maximum opening or gap "W_{max}" is equal to $0.3\ mm$, so I will have to take into account the maximum bending moment for cracking for this size in case of cracking of the element.

The maximum bending moment for cracking for the delta beam is $M_f = 708.20\ kN \cdot m$

Now I have to determine the bending moment applied to the element without enlargement "M_a", in order to check if there is cracking or not:

$M_a \leq M_f \rightarrow$ There is no cracking on the delta beam
$M_a \geq M_f \rightarrow$ There is cracking on the delta beam

In case of cracking, I will have to check that the bending moment applied doesn´t exceed the maximum bending moment for cracking for a crack of $0.3\ mm$.

$$M_d = M_a = \sum \frac{q_i \cdot l^2}{8} = M_{s-w} + M_{sheet} + M_n$$

$$M_{s-w} = \frac{q_{s-w} \cdot l^2}{8} = \frac{3.76 \cdot 14.6^2}{8} = 100.35\ kN \cdot m$$

$$M_{sheet} = \frac{q'_{sheet} \cdot l^2}{8} = \frac{0.3424 \cdot 14.6^2}{8} = 9.12\ kN \cdot m$$

$$M_n = \frac{q'_n \cdot l^2}{8} = \frac{4.8 \cdot 14.6^2}{8} = 127.90\ kN \cdot m$$

So, $M_d = 100.35 + 9.12 + 127.90 = 237.36\ kN \cdot m \leqslant 708.2\ kN \cdot m \rightarrow$ There is no cracking.

As there aren´t any cracks, we can say that $M_a \leq M_f \leq M_{f0.3}$

6.1.2.2 *Verification of the maximum admissible displacement or strain "Δfl_{total}"*

I will consider the following displacements for the delta beam:

- At the arrival of the construction site it will be $0.0\ mm$
- After applying all the loads of the roof it will be $8.0\ mm$
- When time tends to infinite while supporting loads it will be $15.0\ mm$

By using article 50 of the EHE, I will have to verify that $fl_{total} \leq \Delta fl_{total}$, where:

fl_{total} is the total displacement produced on the element during all the steps of the construction.
Δfl_{total} is the maximum total displacement acceptable on the element.

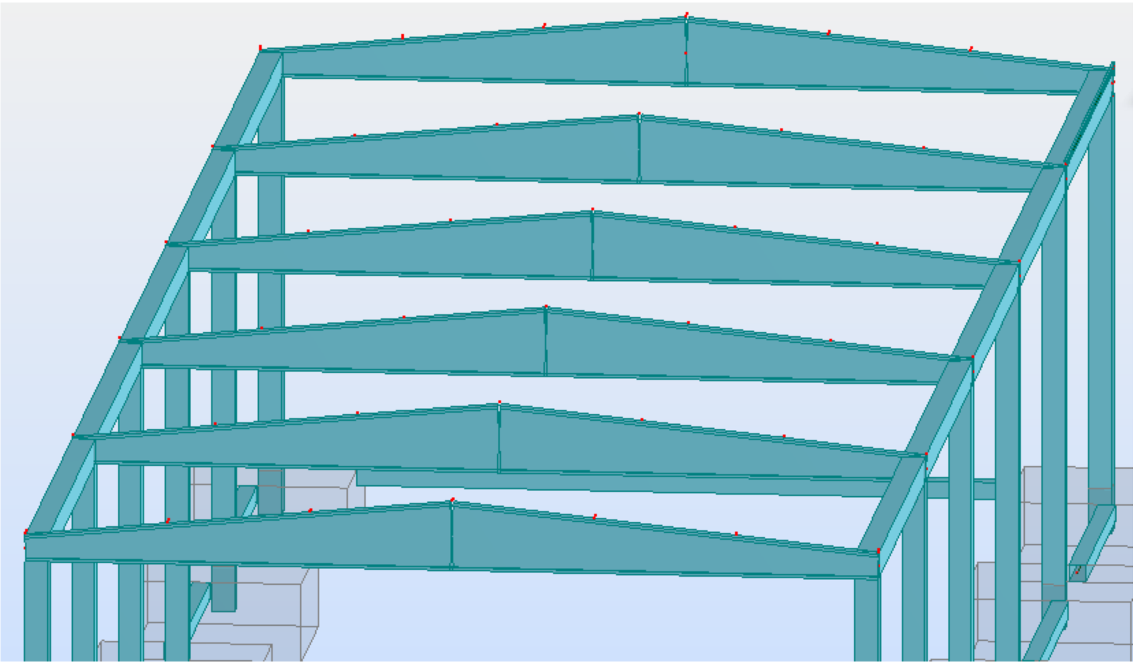
$$fl_{total} = 15.0 - 0.0 = 15\ mm$$

$$\Delta fl_{total} = \min\left(\frac{l}{250}; \frac{l}{500} + 1\right) = \min\left(\frac{1460}{250}; \frac{1460}{500} + 1\right) = \min(5.9; 3.92) = 3.92\ cm \rightarrow$$

$$\Delta fl_{total} = 39.2\ mm$$

$$fl_{total} = 15\ mm \leq 39.2\ mm \rightarrow Verified$$

6.1.3 Final disposition of delta beams



7 COLUMNS

7.1 Introduction

In order to dimension the columns in an appropriate way, I have to take into account the actions that can influence on them. I should take into account permanent and variable actions, self-weight of all the structural elements, wind or snow. But if I include all the self-weights as loads of the previous elements, the only thing that should be calculated is the wind load for each column.

7.2 Action of the wind on the columns

Generally, constructions are not sensible to the dynamic effects of the wind, so I have to check the action of it in all directions in order to determine the load that exerts on the structure, independently of the existence of further buildings in the surroundings.

This load is applied in the outer part of the façade, which will be transmitted to the side columns, as the enclosures doesn´t have to resist anything.

I will calculate it with the same expression as for the roof:

$$q_e = q_b \cdot c_e \cdot c_p$$

q_b dynamic pressure of the wind, with a constant value of $0.5\ kN/m^2$ for all the Spanish territory.

c_e exposure coefficient, dependent on the high of the considered point.

c_p Eolic or pressure coefficient, with a value dependent on the direction of the wind.

7.2.1 Exposure coefficient "c_e"

If we want to calculate c_e for heights of less than 200 m above the terrain, we can use the following formulas:

$$F = k \cdot \ln\left(\frac{\max(z, Z)}{L}\right)$$

$$c_e = F \cdot (F + 7k)$$

The parameters k, L, and Z are obtained from table D.2 of the Basic Document SE-AE:

Grado de aspereza del entorno		Parámetro		
		k	L (m)	Z (m)
IV	Zona urbana en general, industrial o forestal	0,22	0,3	5,0

So, I have $k = 0.22$, $L = 0.3\ m$ and $Z = 5.0\ m$

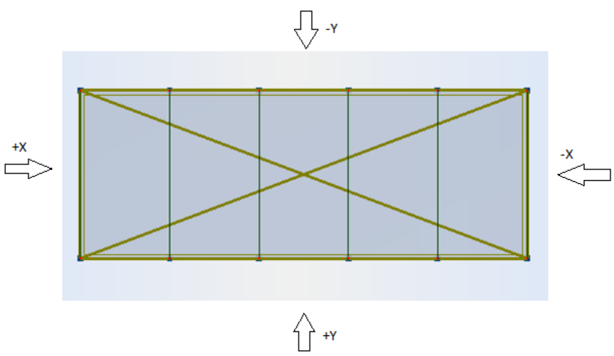
I will take as limit values $z = 0$ m and $z = 9\ m$, because the largest value should be $8.2\ metres$, and like that I take more security margin:

Height z	max (z,Z)	F=k•ln(max(z,Z)/L)	ce=F•(F+7k)
0	5	0,61895036	1,336283096
1	5	0,61895036	1,336283096
2	5	0,61895036	1,336283096
3	5	0,61895036	1,336283096
4	5	0,61895036	1,336283096
5	5	0,61895036	1,336283096
6	6	0,6590611	1,449315628
7	7	0,69297425	1,547393655
8	8	0,72235116	1,634211973
9	9	0,74826342	1,712223825

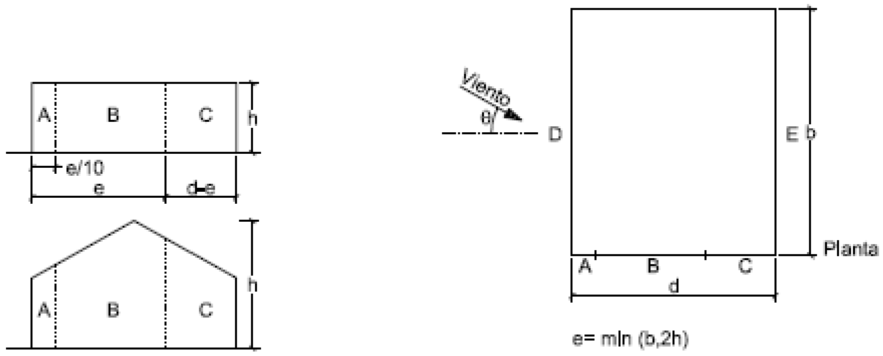
As we can see, c_e will be constant until $z = 5\ m$, and then will begin to increase progressively, exerting each time more load on the structure.

7.2.2 Eolic or pressure coefficient “c_p”

The Eolic or pressure coefficient is variable depending on the direction of the wind, so there are lots of possibilities and it is impossible to evaluate all of them.
According to the Basic Document SE-AE, it will be enough to evaluate just two directions visibly orthogonal, considering both senses for each.
Here we can see the directions considered:

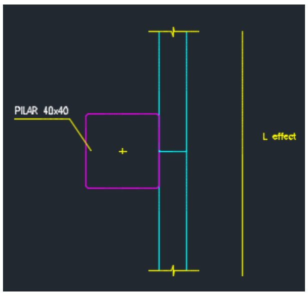


Wind only acts on the façade, so I can obtain these coefficients by table D.3 from the Basic Document SE-AE. Positive values indicate pressure, and negative values indicate suction.



A (m^2)	h/d	Zona (según figura), -45º < θ < 45º A	B	C	D	E
≥ 10	5	-1,2	-0,8	-0,5	0,8	-0,7
	1	"	"	"	"	-0,5
	≤ 0,25	"	"		0,7	-0,3

a is the area of influence of the column considered. It is obtained by multiplying the height of the façade and half of the distance between previous and post columns. In my case, columns are separated $8\ metres$, so this area would be:



$a = \left(\frac{l}{2}+\frac{l}{2}\right)\cdot h = \left(\frac{8}{2}+\frac{8}{2}\right)\cdot 8.2 = 65.6\ m^2$, or $a/2$ for the corner ones, $32.8\ m^2$.

h is the height of the façade. In my case, 8.2 metres.

d is the depth of the building in the direction considered. For direction X is $40.4\ metres$, and for direction Y is $15\ metres$.

As +Y and –Y have h/d different from the ones of the table, I have to interpolate in order to obtain its values

Direction	Depth "d" (m)	Height "h" (m)	h/d	Pressure coefficient	Suction coefficient
(+X)	40,4	8,2	0,202970297	0,7	-0,3
(+Y)	15	8,2	0,546666667	0,73955556	-0,37911111
(-X)	40,4	8,2	0,202970297	0,7	-0,3
(-Y)	15	8,2	0,546666667	0,73955556	-0,37911111

The worst q_e will be the one which comes from the highest c_e and the highest c_p, for each direction. So, for

+X we have $q_e = 50 \cdot 1{,}7122 \cdot 0.7 = 59.93\ \frac{kg}{m^2}$ pressure

-X we have $q_e = 50 \cdot 1{,}7122 \cdot 0.3 = 25.7\ \frac{kg}{m^2}$ suction

+Y we have $q_e = 50 \cdot 1{,}7122 \cdot 0.7400 = 63.3\ \frac{kg}{m^2}$ pressure

-Y we have $q_e = 50 \cdot 1{,}7122 \cdot 0.3791 = 32.45\ \frac{kg}{m^2}$ suction

Here I plot the values for each direction:

Direction	Height z	c_e	Pressure coefficient	Suction coefficient	$q_e = q_b \cdot c_e \cdot c_p$	
(+X)	0-5	1,336283096	0,7		46,7699084	
				0,3	20,0442464	
	6	1,449315628	0,7		50,726047	
				0,3	21,7397344	
	7	1,547393655	0,7		54,1587779	kg/m^2
				0,3	23,2109048	
	8	1,634211973	0,7		57,1974191	
				0,3	24,5131796	
	9	1,712223825	0,7		59,9278339	
				0,3	25,6833574	

Direction	Height z	c_e	Pressure coefficient	Suction coefficient	$q_e = q_b \cdot c_e \cdot c_p$	
(-X)	0-5	1,336283096	0,7		46,7699084	
				0,3	20,0442464	
	6	1,449315628	0,7		50,726047	
				0,3	21,7397344	
	7	1,547393655	0,7		54,1587779	kg/m^2
				0,3	23,2109048	
	8	1,634211973	0,7		57,1974191	
				0,3	24,5131796	
	9	1,712223825	0,7		59,9278339	
				0,3	25,6833574	

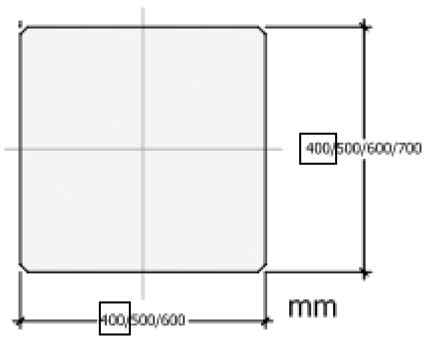
Direction	Height z	c_e	Pressure coefficient	Suction coefficient	$q_e = q_b \cdot c_e \cdot c_p$	
(+Y)	0-5	1,336283096	0,73955556		49,4127794	kg/m^2
				-0,379111111	-25,3299885	
	6	1,449315628	0,73955556		53,5924712	
				-0,379111111	-27,4725829	
	7	1,547393655	0,73955556		57,2191787	
				-0,379111111	-29,3317064	
	8	1,634211973	0,73955556		60,4295272	
				-0,379111111	-30,9773958	
	9	1,712223825	0,73955556		63,3142321	
				-0,379111111	-32,4561538	

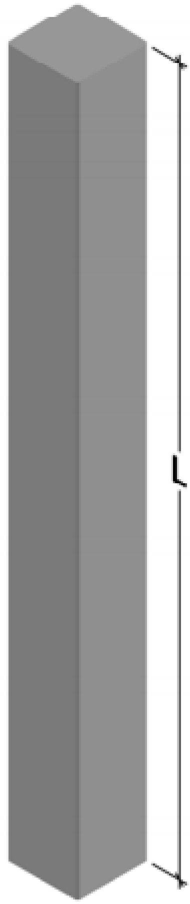
Direction	Height z	c_e	Pressure coefficient	Suction coefficient	$q_e = q_b \cdot c_e \cdot c_p$	
(-Y)	0-5	1,336283096	0,73955556		49,4127794	kg/m^2
				-0,379111111	-25,3299885	
	6	1,449315628	0,73955556		53,5924712	
				-0,379111111	-27,4725829	
	7	1,547393655	0,73955556		57,2191787	
				-0,379111111	-29,3317064	
	8	1,634211973	0,73955556		60,4295272	
				-0,379111111	-30,9773958	
	9	1,712223825	0,73955556		63,3142321	
				-0,379111111	-32,4561538	

7.3 Calculation of columns

7.3.1 Type of column

The type of column I will be using will be this one:





PESO (KN/m)						
40X40	40X50	50X50	50X60	50X70	60X60	60X70
4,00	5,00	6,25	7,50	8,75	9,00	10,50

Other values of interest:

$q_{s-w\ column} = 3120\ kg;$ $L = 7.8\ m;$ $M_u = 590\ kN \cdot m;$

$V_{u1} = 1440\ kN;$ $d = 400\ mm;$ $s_t = 120\ mm;$

$V_{u2} = 334\ kN;$ $W_k = 0.38\ mm;$

7.3.2 Loads to consider

I will have to consider different loads depending on the column where I am calculating actions. For corner columns, I will have to take into account the self-weight of the roof and the purlins, the action of the snow, the self-weight of the concrete channel and the self-weight of the delta beam.

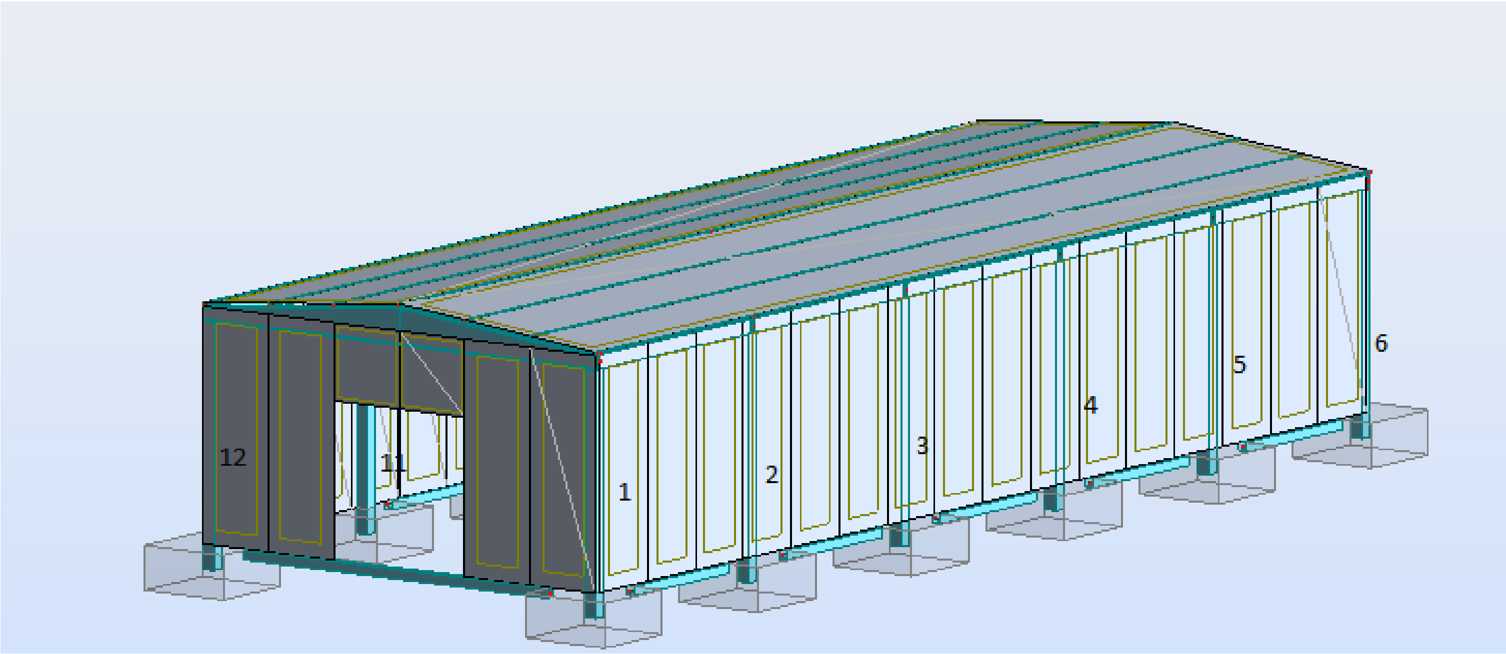
For inner columns, loads will be the same. The difference will be that they will have double area of action, as they will have structure before and after them. We will see this later in a better way.

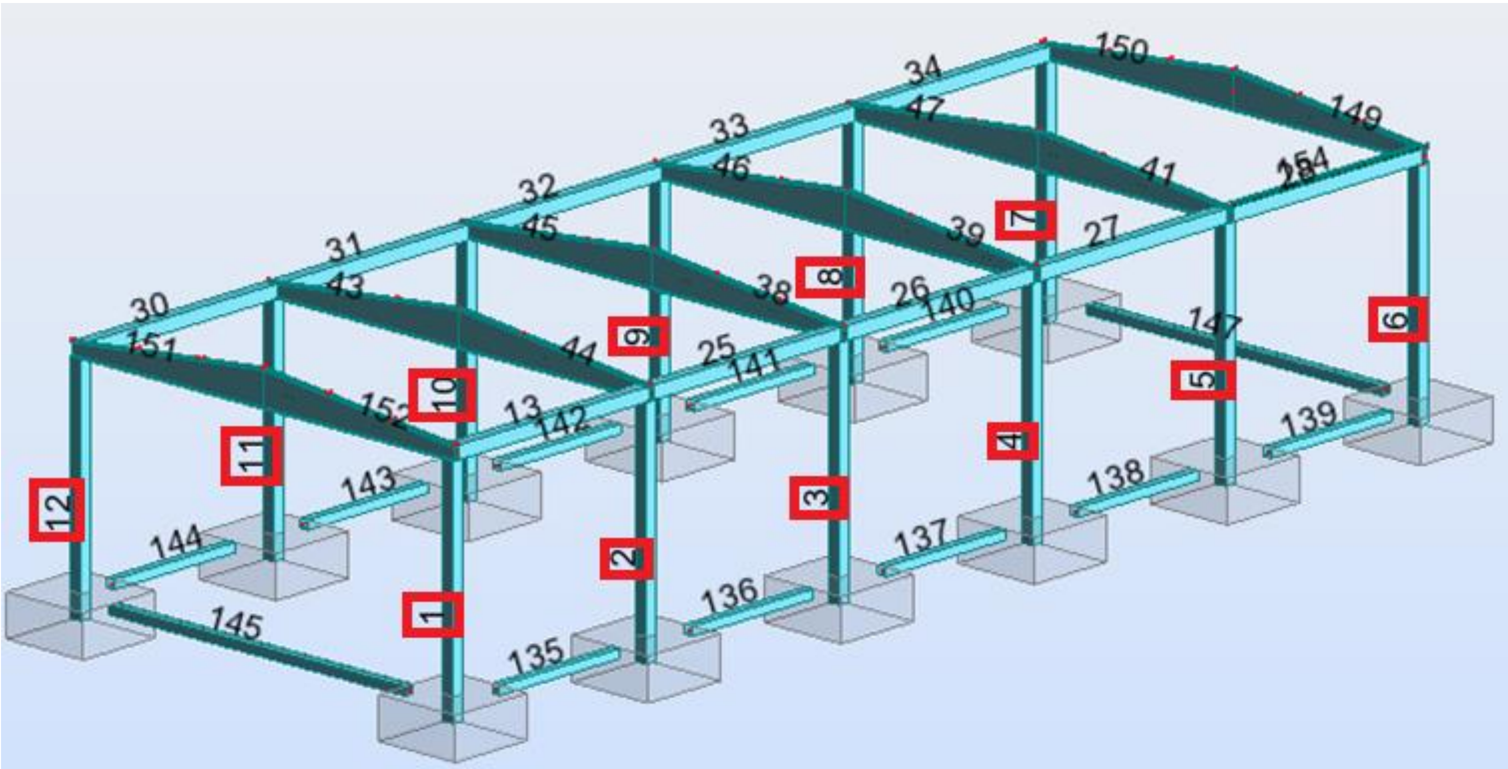
7.3.3 Hypothesis to consider

I will consider different hypothesis of loading, in order to obtain the worst for the dimensioning.

1. Permanent loads + overloads
2. Permanent loads + wind in direction +X
3. Permanent loads + wind in direction –X
4. Permanent loads + wind in direction +Y
5. Permanent loads + wind in direction –Y
6. Permanent loads + overloads + 0.6 · wind in direction +X
7. Permanent loads + overloads + 0.6 · wind in direction –X
8. Permanent loads + overloads + 0.6 · wind in direction +Y
9. Permanent loads + overloads + 0.6 · wind in direction –Y
10. Permanent loads + 0.7 · overloads + wind in direction +X
11. Permanent loads + 0.7 · overloads + wind in direction –X
12. Permanent loads + 0.7 · overloads + wind in direction +Y
13. Permanent loads + 0.7 · overloads + wind in direction –Y

7.3.4 Name of the columns





7.3.5 Cases to consider

7.3.5.1 Corner columns

Self-weight of concrete channel: $\frac{q_{s-w\ channel}}{supports} = \frac{2096}{2} = 1048\ kg = q'_{channel}$

Self-weight of the column: $q_{s-w\ column} = 3120\ kg = q'_{column}$

Self-weight of delta beam: $\frac{q_{s-w\ dbeam}}{supports} = \frac{5649}{2} = 2824.5\ kg = q'_{dbeam}$

Self-weight of roof + purlins: $(q_{sheet} + q_{purlin}) \cdot l_{interaxis} \cdot \left(\frac{l_{dbeam}}{supports}\right) = (4.28 + 3.08) \cdot 4 \cdot \left(\frac{15}{2}\right) = 220.8\ kg = q'_{sheet+purlin}$

Self-weight snow: $q_n \cdot l_{interaxis} \cdot \left(\frac{l_{dbeam}}{supports}\right) = 60 \cdot 4 \cdot \left(\frac{15}{2}\right) = 1800\ kg = q'_n$

- Permanent loads centred in the axis:
    - Axial: $F1 = q'_{channel} + q'_{column} = 1048 + 3120 = 4168\ kg = 41.68\ kN$
    - Bending moment: $M1 = 0$ (F1 located in the centre of the column).
- Permanent loads out of the axis:
    - Axial: $F2 = q'_{dbeam} + q'_{sheet+purlin} = 2824.5 + 220.8 = 3045.3\ kg = 30.45\ kN$
    - Bending moment: $M2 = F2 \cdot d_{axis} = 30.45 \cdot 0.2 = 6.09\ kN \cdot m$, with $d_{axis} = \frac{l_{support}}{2} = 0.4/2 = 0.2\ m$
- Total permanent loads:
    - Axial: $Ft = F1 + F2 = 4168 + 3045.3 = 7213.3\ kg = 72.13\ kN$
    - Bending moment: $Mt = M1 + M2 = 0 + 6.09 = 6.09\ kN \cdot m$
- Overloads with roof and delta beam (I take into account snow load as overload):
    - Axial: $F3 = q'_n = 1800\ kg = 18\ kN$
    - Bending moment: $M3 = F3 \cdot d_{axis} = 18 \cdot 0.2 = 3.6\ kN \cdot m$
- Wind action:
  Depends on the direction of the wind. For each hypothesis I will have different values.
  Direction +X: Shear: $V_{+X} = \sum q_{e+X} = 45.59\ kN$
  Bending moment: $M_{+X} = \sum(q_{e+X} \cdot d_0) = 238.19\ kN \cdot m$

Direction -X: Shear: $V_{-X} = \sum q_{e-X} = 45.59\ kN$
Bending moment: $M_{-X} = \sum (q_{e-X} \cdot d_0) = 238.19\ kN \cdot m$

Direction +Y: Shear: $V_{+Y} = \sum q_{e+Y} = 48.16\ kN$
Bending moment: $M_{+Y} = \sum (q_{e+Y} \cdot d_0) = 251.66\ kN \cdot m$

Direction -Y: Shear: $V_{-Y} = \sum q_{e-Y} = 48.16\ kN$
Bending moment: $M_{-Y} = \sum (q_{e-Y} \cdot d_0) = 251.66\ kN \cdot m$

1. Results for column 1:

Column 1			
Hypothesis	Axial stress [KN]	Shear stress [KN]	Bending moment [KN·m]
1	90,133	0	9,6906
2	72,133	45,58596197	244,285221
3	72,133	0	6,0906
4	72,133	48,16193061	257,745108
5	72,133	0	6,0906
6	90,133	27,35157718	152,607373
7	90,133	0	9,6906
8	90,133	28,89715837	160,683305
9	90,133	0	9,6906
10	84,733	45,58596197	246,805221
11	84,733	0	8,6106
12	84,733	48,16193061	260,265108
13	84,733	0	8,6106

2. Results for column 6:

Column 6			
Hypothesis	Axial stress [KN]	Shear stress [KN]	Bending moment [KN·m]
1	90,133	0	12,1812
2	72,133	0	6,0906
3	72,133	45,58596197	244,285221
4	72,133	48,16193061	257,745108
5	72,133	0	6,0906
6	90,133	0	9,6906
7	90,133	27,35157718	152,607373
8	90,133	28,89715837	160,683305
9	90,133	0	9,6906
10	84,733	0	8,6106
11	84,733	45,58596197	246,805221
12	84,733	48,16193061	260,265108
13	84,733	0	8,6106

3. Results for column 7:

Column 7			
Hypothesis	Axial stress [KN]	Shear stress [KN]	Bending moment [KN·m]
1	90,133	0	12,1812
2	72,133	0	6,0906
3	72,133	45,58596197	244,285221
4	72,133	0	6,0906
5	72,133	48,16193061	257,745108
6	90,133	0	9,6906
7	90,133	27,35157718	152,607373
8	90,133	0	9,6906
9	90,133	28,89715837	160,683305
10	84,733	0	8,6106
11	84,733	45,58596197	246,805221
12	84,733	0	8,6106
13	84,733	48,16193061	260,265108

4. Results for column 12:

Column 12			
Hypothesis	Axial stress [KN]	Shear stress [KN]	Bending moment [KN·m]
1	90,133	0	12,1812
2	72,133	45,58596197	244,285221
3	72,133	0	6,0906
4	72,133	0	6,0906
5	72,133	48,16193061	257,745108
6	90,133	27,35157718	152,607373
7	90,133	0	9,6906
8	90,133	0	9,6906
9	90,133	28,89715837	160,683305
10	84,733	45,58596197	246,805221
11	84,733	0	8,6106
12	84,733	0	8,6106
13	84,733	48,16193061	260,265108

7.3.5.2 Inner columns

Self-weight of concrete channel: $\frac{q_{s-w\ channel}}{supports} \cdot sides = \frac{2096}{2} \cdot 2 = 2096\ kg = q'_{channel}$

Self-weight of the column: $q_{s-w\ column} = 3120\ kg = q'_{column}$

Self-weight of delta beam: $\frac{q_{s-w\ dbeam}}{supports} = \frac{5649}{2} = 2824.5\ kg = q'_{dbeam}$

Self-weight of roof + purlins: $(q_{sheet} + q_{purlin}) \cdot l_{intercolumns} \cdot \left(\frac{l_{dbeam}}{supports}\right) = (4.28 + 3.08) \cdot 8 \cdot \left(\frac{15}{2}\right) = 441.6\ kg = q'_{sheet+purlin}$

Self-weight snow: $q_n \cdot l_{intercolumns} \cdot \left(\frac{l_{dbeam}}{supports}\right) = 60 \cdot 8 \cdot \left(\frac{15}{2}\right) = 3600\ kg = q'_n$

- Permanent loads centred in the axis:
  - Axial: $F1 = q'_{channel} + q'_{column} = 2096 + 3120 = 5216\ kg = 52.16\ kN$
  - Bending moment: $M1 = 0$ (F1 located in the centre of the column).
- Permanent loads out of the axis:
  - Axial: $F2 = q'_{dbeam} + q'_{sheet+purlin} = 2824.5 + 441.6 = 3266.1\ kg = 32.66\ kN$
  - Bending moment: $M2 = F2 \cdot d_{axis} = 32.66 \cdot 0.2 = 6.53\ kN \cdot m$, with $d_{axis} = \frac{l_{support}}{2} = 0.4/2 = 0.2\ m$
- Total permanent loads:
  - Axial: $Ft = F1 + F2 = 5216 + 3266.1 = 8482.1\ kg = 84.82\ kN$
  - Bending moment: $Mt = M1 + M2 = 0 + 6.53 = 6.53\ kN \cdot m$
- Overloads with roof and delta beam (I take into account snow load as overload):
  - Axial: $F3 = q'_n = 3600\ kg = 36\ kN$
  - Bending moment: $M3 = F3 \cdot d_{axis} = 36 \cdot 0.2 = 7.2\ kN \cdot m$
- Wind action:
  Depends on the direction of the wind. For each hypothesis I will have different values.
  Direction +X: Shear: $V_{+X} = \sum q_{e+X} = 45.59\ kN$
  Bending moment: $M_{+X} = \sum(q_{e+X} \cdot d_0) = 238.19\ kN \cdot m$

  Direction -X: Shear: $V_{-X} = \sum q_{e-X} = 45.59\ kN$
  Bending moment: $M_{-X} = \sum(q_{e-X} \cdot d_0) = 238.19\ kN \cdot m$

Direction +Y: Shear: $V_{+Y} = \sum q_{e+Y} = 48.16\ kN$
Bending moment: $M_{+Y} = \sum (q_{e+Y} \cdot d_0) = 251.66\ kN \cdot m$

- Direction -Y: Shear: $V_{-Y} = \sum q_{e-Y} = 48.16\ kN$
Bending moment: $M_{-Y} = \sum (q_{e-Y} \cdot d_0) = 251.66\ kN \cdot m$

5. Results for columns 2,3,4,5:

Columns 2,3,4,5			
Hypothesis	Axial stress [KN]	Shear stress [KN]	Bending moment [KN·m]
1	120,821	0	13,7322
2	84,821	0	6,5322
3	84,821	0	6,5322
4	84,821	48,16193061	258,186708
5	84,821	0	6,5322
6	120,821	0	13,7322
7	120,821	0	13,7322
8	120,821	28,89715837	164,724905
9	120,821	0	13,7322
10	110,021	0	11,5722
11	110,021	0	11,5722
12	110,021	48,16193061	263,226708
13	110,021	0	11,5722

6. Results for columns 8,9,10,11:

Columns 8,9,10,11			
Hypothesis	Axial stress [KN]	Shear stress [KN]	Bending moment [KN·m]
1	120,821	0	13,7322
2	84,821	0	6,5322
3	84,821	0	6,5322
4	84,821	0	6,5322
5	84,821	48,16193061	258,186708
6	120,821	0	13,7322
7	120,821	0	13,7322
8	120,821	0	13,7322
9	120,821	28,89715837	164,724905
10	110,021	0	11,5722
11	110,021	0	11,5722
12	110,021	0	11,5722
13	110,021	48,16193061	263,226708

According to the data previously calculated, the worst hypothesis for the industrial building would be hypothesis 13 for columns 8, 9, 10 and 11. In this case we have $Bending\ moment\ in\ X = 110.02\ kN \cdot m$, $Shear\ stress\ in\ Y = 48.16\ kN$ and $Total\ Axil = 263.23\ kN$.

7.3.6 Verification of the Ultimate Limit State (ULS)

As Ultimate Limit State I will to check the following values:

Applied bending moment "M_d"

Applied shear stress "V_d"

For this verification the next condition must be satisfied:

$S_d \leq R_d$ being S_d the value calculated for the effect of the actions

R_d the value calculated for the structural behaviour.

In order to verify the ULS in a correct way, the bending moment and the shear stress must be taken with enlargement factors "γ". This means that for permanent actions I should take $\gamma = 1.35$, and for variable actions it should be $\gamma = 1.5$. As we cannot use this values, because in the columns I have both permanent and variable actions, I will use an enlargement factor as a mean value between the first and the second value.

So, I will take the enlargement factor as $\gamma_{aprox} = 1.42$.

7.3.6.1 Verification of the bending moment applied "M_d"

According to the technical data of the columns:

$$M_u = 590\ kN \cdot m$$

So, the condition to verify will be $M_d \leq M_u$, being M_d the maximum bending moment created by the actions on the column.

As I have previously calculated:

$$M_a = 263.22\ kN \cdot m$$

Applying the enlargement factor $\gamma_{aprox} = 1.42$:

$M_d = M_a \cdot \gamma_{aprox} = 263.22 \cdot 1.42 = 373.78\ kN \cdot m.$

$$M_d = 373.78\ kN \cdot m \leq M_u = 590\ kN \cdot m \rightarrow Verified.$$

7.3.6.2 *Verification of the shear stress applied "V_d"*

In order to be able to consider the help of the reinforcement (shear) of the column in the determination of the ultimate shear stress, I have to meet the next conditions from EHE-08 for the distance between transversal reinforcement "s_t". I will do this in the bottom of the column, as is there where all the forces are applied:

$s_t \leqslant 0.75 \cdot d \cdot (1 + cotg\alpha)$ and $s_t \leqslant 600\ mm$

If $V_d \leqslant \frac{1}{5} \cdot V_{ul}$

where $V_{u1} = 1440\ kN;$ $d = 400\ mm;$ $\alpha = 90°;$

$$V_d = V_a \cdot \gamma_{aprox} = 48.16 \cdot 1.42 = 68.39\ kN$$

$s_t = 120\ mm$

$V_d \leqslant 1/5 \cdot V_{u1} \rightarrow 68.39 \leqslant 288\ kN$ → satisfied

$s_i \leqslant 0.75 \cdot d \cdot (1 + cotg\alpha) \rightarrow\ s_i \leqslant 300\ mm$ → satisfied

$s_i \leqslant 600\ mm$ → satisfied

This means that I can take into account the shear reinforcement as the distance is lower than the admissible.

The data for the column are:
$V_{u1} = 1440\ kN$ → fatigue shear due to compression of the web.

$V_{u2} = 334\ kN$ → fatigue shear due to stress of the web.

For an structural element with shear reinforcement I have to verify that:
$V_d \leqslant V_{u1}$ and $V_d \leqslant V_{u2}$

$$V_d = 68.39 \leq 334 \leq 1440\ kN \rightarrow Verified$$

7.3.7 Verification of the Serviceability Limit State (SLS)

For the verification of the Serviceability Limit State, I will verify:

- Cracking bending moment "M_f"
- Horizontal displacement "Δd_y" in Y direction

When checking these limit states the following condition must be satisfied:

$E_d \leq C_d$ being E_d value of the effect of the actions

C_d limit value admissible for the section

7.3.7.1 Verification of the cracking bending moment "M_f"

According to the data previously calculated:

$$M_a = 263.23\ kN \cdot m$$

Taking into account the technical data for the columns, I have:

$$W_k = 0.38\ mm$$

According to table 8.2.2 from the EHE-08, the structural elements protected against weather conditions are included in the Exposition class of type I:

Clase General de Exposición				DESCRIPCIÓN	EJEMPLOS
Clase	*Subclase*	*Designación*	*Tipo de proceso*		
No agresiva		I	Ninguno	Interiores de edificios no sometidos a condensaciones	Elementos estructurales protegidos de la intemperie

If we go to table 5.1.1.2 of the EHE-08, for the type of element that we have, the maximum width for the crack," W_{max}" will be $0.4\ mm$:

Tabla 5.1.1.2

Clase de exposición, según artículo 8°	$w_{máx}$ [mm]	
	Hormigón armado (para la combinación cuasipermanente de acciones)	Hormigón pretensado (para la combinación frecuente de acciones)
I	0,4	0,2

So:

$$W_k \leq W_{max} \rightarrow 0.38 \leq 0.4\ mm \rightarrow M_f \leq M_a \leq M_{f0.4}\ kN \cdot m$$

With this, it is verified that actually there is cracking and that the element stands the cracking bending moment applied, as the crack is smaller than the admissible one.

7.3.7.2 Verification of the horizontal displacement "Δd_y" in Y direction

The horizontal displacement will be:

$$\Delta d_y = 15\ mm$$

The Basic Document SE indicates in point 4.3.3.2 that under any kind of combination of characteristic actions or hypothesis, the total collapse must be $\Delta d_y \leq \frac{1}{500} \cdot h_{element}$, being $h_{element}$ the height of the element of study, which for this case it will be 7.8 m.

$$\Delta d_y \leq 1/500 \cdot h_{element} \rightarrow 15 \leq \frac{1}{500} \cdot 7.8 \rightarrow 15 \leq 15.6\ mm \rightarrow Verified$$

8 CALCULATION OF THE FOUNDATIONS OF THE INDUSTRIAL BUILDING

8.1 Type of footings

The foundation of the building will be made by isolated rigid footings, which will be supporting the columns for the building. They will have attachment beams between them, giving more stability and supporting the enclosures. The methodology for the footings will be by calyx method, like in the picture:

They will have to be designed according to the requirements of the building.

Other values of interest:

$f_{ck} = 250\ kg/cm^2$; $\gamma_c = 1.5$; $f_{yk} = 5100\ kg/cm^2$;

$\gamma_s = 1.15$; $\gamma_H = 2500\ kg/m^3$; $c = 3\ cm$

8.2 General recommendations for the footings

For the calculations I will use the Basic Document SE-C (Foundations) of the CTE.

As constructive guidance, and for the construction of the footings, I will have into account the following recommendations:

- Under the footings I will place levelling or poor concrete, which resistance is $20\ N/mm^2$, up to a depth which will depend on each footing. It will be used to get to the terrain which is the recommended for the footings. The minimum thickness for this concrete will be $5\ cm$.
- The maximum distance between reinforcements won´t be greater than $30\ cm$ or smaller than $10\ cm$. If needed, they will be placed by pairs.

- Diameters smaller than 12 mm won´t be used for the bottom bars, and the steel won´t be worse than B 400 S.
- The lateral covering for the head of the bars cannot be less than 5 cm, in order to assure that the bars will fit in the wholes with normal excavation and cutting tolerances .
- It is recommended to manage the horizontal dimensions in multiples of 25 cm, and the edges in multiples of 10 cm, in order to ease the process.
- The geometric amount for the reinforcements in each direction, for steel B 500 S, is of $0.0009\ kg$.
- I will use longitudinal coverings greater than 3 cm, because when making the footing on levelling concrete we don´t have the obligation of fitting 3.7.2.4.1 from the EHE-08. This says that we must have a covering greater than 7 cm in pieces concreted to the terrain, unless it is done on levelling concrete.
- The resistance of the terrain, or maximum admissible strain, will be 2.2 kg/cm^2.

The elements I will use for the construction of the footing will be:

- Concrete: $f_{ck} = 250\ kg/cm^2$. The security coefficient I will use for this concrete will be $\gamma_c = 1.5$.
- Steel: $f_{yk} = 5100\ kg/cm^2$. The security coefficient I will use for the steel will be $\gamma_s = 1.15$.

8.3 Calculation of the footings

8.3.1 Actions to consider and initial dimensioning of the footing

I will calculate the footings taking into account the same hypothesis considered for the calculation of the columns, this is, the most unfavourable ones. Its values are:

$$M_a = Bending\ moment\ in\ X = 263.23\ kN \cdot m$$

$$V_a = Shear\ stress\ in\ Y = 48.16\ kN$$

$$N_a = Total\ Axil = 110.02\ kN$$

The first step for designing and calculating the footing is to do an initial dimensioning of it, and perform after that some checking's that will tell us if these values are acceptable. I will take the following values as reference:

$a = 3\ m$
$b = 3\ m$
$h = 1.40\ m$

This is a footing with a square top view and a height of 1.40 metres.

8.3.2 Checking of the rollover

Rollover is a failure mode for foundations that have to stand horizontal loads and big bending moments when, being small the width of the footing, the predominant movement is the bending of it. In order to check that the footing chosen in the previous step is enough to stand the rollover I have to compare stabilizing bending moments "M_1" with those which tend to cause the rollover of the footing "M_2". This will be done by meeting the next condition:

$$M_1 \geq M_2 \rightarrow (N_a + P) \cdot \frac{a}{2} \geq (M_a + V_a \cdot h) \cdot \gamma_E$$

Where:

γ_E → Security coefficient for the rollover. Its value is $\gamma_E = 1.8$ for destabilizing actions, according to table 2.1 from the Basic Document SE-C:

Tabla 2.1. Coeficientes de seguridad parciales

Situación de dimensionado	Tipo	Materiales		Acciones	
		γ_R	γ_M	γ_E	γ_F
	Acciones desestabilizadoras	1,0	1,0	1,8	1,0

P → Self-weight of the footing. Taking into account that the specific weight of the concrete is $\gamma_H = 2500\ kg/m^3$:

$$P = \gamma_H \cdot Volume = 2500 \cdot 3 \cdot 3 \cdot 1.40 = 31500\ kg \approx 315\ kN$$

So we have the following values:

$$M_1 = (110.02 + 315) \cdot \frac{3}{2} = 637.53\ kN \cdot m$$

$$M_2 = (263.23 + 48.16 \cdot 1.40) \cdot 1.8 = 595.18\ kN \cdot m$$

The condition becomes:

$$M_1 \geq M_2 \rightarrow 637.53 \geq 595.18\ kN \cdot m \rightarrow Verified$$

8.3.3 Checking of the condition of rigid footing

Now, I will check if with the dimensions previously chosen the condition of rigid footing is satisfied. This means that the distance between the edge of the footing and the beginning of the column is less than double of the height of the footing:

$$Distance < 2 \cdot h$$

$$Distance = \frac{a - x}{2} = \frac{300 - 40}{2} = 130\ cm$$

$$2 \cdot h = 2 \cdot 140 = 280\ cm$$

$$Distance < 2 \cdot h \rightarrow 130 < 280\ cm \rightarrow Verified$$

8.3.4 Checking of the sliding

As there will be attachment beams between the footings, in order to support the self-weight of the enclosures and give more stability, this condition will be directly satisfied.

8.3.5 Checking of the collapse of the terrain or of the admissible strain of the terrain "$\sigma_{terrain}$"

The first step to perform in the verification of the resistance of the terrain against the strains required is to determine the eccentricity with which vertical efforts act on the footing, classifying in this way the type of distribution of the loads which fits our case, depending on:

- Case 1: $e < \frac{a}{6} \rightarrow \sigma_1 and\ \sigma_2$ compressive stress.
- Case 2: $e = \frac{a}{6} \rightarrow \sigma_2 = 0$ and σ_1 is compressive stress.
- Case 3: $e > a/6 \rightarrow \sigma_2$ is traction strain and σ_1 is compressive stress.

For the calculation of the eccentricity I will have to take into account the self-weight of the footing, as it has quite big dimensions:

$$e = \frac{\sum Bending\ moments}{\sum Axil\ moments} = \frac{(M_a + V_a \cdot h)}{(N_a + P)} = \frac{(263.23 + 48.16 \cdot 1.40)}{(110.02 + 315)} = \frac{330.65}{425.02} = 0.78\ m$$

$$\frac{a}{6} = \frac{3}{6} = 0.5\ m$$

As the eccentricity "e" is greater than $a/6$, the distribution of stress is the one of case 3, being necessary to verify them by the triangular law.

Triangular law:

$$\sigma_1 \leq 1.25 \cdot \sigma_{adm}$$

$\sigma_{adm} = 2.20\ kg/cm^2 \rightarrow$ Admissible strain indicated for this kind of terrain

$$C = \frac{a}{2} - e = \frac{300}{2} - 78.8 = 72.2\ cm$$

$$\sigma_1 = \frac{2 \cdot (N_a + P)}{3 \cdot C \cdot b} = \frac{2 \cdot (110.02 + 315)}{3 \cdot 72.2 \cdot 300} = \frac{850.04}{64980} = 0.0131\frac{kN}{cm^2} = 1.31\frac{kg}{cm^2}$$

With these two values:

$$\sigma_1 \leq 1.25 \cdot \sigma_{adm} \rightarrow 1.31 \leq 1.25 \cdot 2.2 \rightarrow 1.31 \leq 2.75\frac{kg}{cm^2} \rightarrow Verified$$

8.3.6 Calculation of the principal reinforcement

According to article 58.2.1 of the EHE-08, in rigid foundations, as we have checked before, the distribution of the strain is not lineal. So, the best method for the general analysis is the connection rods and cables method (Método de bielas y tirantes).

This method consists in substituting the structure by a structure made of articulated bars which represent its behaviour. The compressed bars are denominated connection rods, and represent the compression of the concrete “R_d”. The bars tensioned are called cables, and represent the stress forces on the reinforcement “T_d”.

The first step to perform to calculate the principal reinforcement of the footing is to determine the eccentricity with which act the enlarged vertical efforts, in order to be able to apply:

- Case 1: $e_d < \frac{a}{3} \rightarrow x_1 \approx 0.3 \cdot a\ ;\ R'_{1d} = \frac{\sigma_{1d}+\sigma_{3d}}{4} \cdot a \cdot b$
  being “σ_{3d}” the compressive effort of the triangle produced by the axis of the column.
- Case 2: $e_d > \frac{a}{3} \rightarrow x_1 = e_d\ ;\ R'_{1d} = N_d$

For the calculation of the eccentricity produced by the enlarged actions I mustn´t take into account the self-weight of the footing, so its value will be:

$$e_d = \frac{\sum Bending\ moments}{N_d} = \frac{(M_d + V_d \cdot h)}{(N_a \cdot \gamma_{aprox})} = \frac{373.78 + 68.39 \cdot 1.4}{110.02 \cdot 1.42} = \frac{469.53}{156.23} = 3.01m$$

Where:
M_d → Calculated bending moment exerted by the actions, previously calculated.
V_d → Calculated shear exerted by the actions, previously calculated.
γ_{aprox} → Approximated enlargement coefficient, previously calculated.

$$\frac{a}{3} = \frac{3}{6} = 0.5\ m$$

As eccentricity “e_d” is larger than $a/3$, I will use case 2 to perform the calculation, applying the equation indicated in point 5.8.4.1 from the EHE-08 for rigid foundations:

$$T_d = \frac{R'_{1d}}{0.85 \cdot d} \cdot (x_1 - 0.25 \cdot a) = A_s \cdot f_{yd}$$

Where:

d → Useful edge of the footing, this means, distance from the bottom part of the column to the passive reinforcement for stress located on the bottom part of the footing. Taking into account that in the first part of the footings I said that the minimum covering for the reinforcement would be 3 cm:

$$d = h - (z + c) = 140 - (80 + 3) = 57\ cm$$

c → Covering of the footing
z → Submerged part of the column into the footing, which is 80 cm in this case

R'_{1d} → Has the same value as $N_d = 156.23\ kN$

x_1 → Has the same value as $e_d = 3.01\ m$

A_s → Total area of the passive reinforcement to stress

f_{yd} → Calculation resistance of the steel of the passive reinforcement to stress. Its value is obtained from:

$$f_{yd} = \frac{f_{yk}}{\gamma_s} = \frac{5100}{1.15} = 4434.78\,\frac{kg}{cm^2} \approx 44.35\ kN/cm^2$$

With $f_{yk} = 5100\,\frac{kg}{cm^2}$ and $\gamma_s = 1.15$, previously defined.

Introducing in the first equation these values:

$$T_d = \frac{156.23}{0.85 \cdot 0.57} \cdot (301 - 0.25 \cdot 300) = A_s \cdot 44.35 \rightarrow T_d = 727.26 = A_s \cdot 44.35 \rightarrow$$

$$A_s = \frac{727.26}{44.35} = 16.40\ cm^2$$

Now I will check the limitations according to the minimum geometric amount, imposed by the EHE-08 in table 42.3.5:

Tipo de elemento estructural	Tipo de acero	
	Aceros con $f_y = 400\ N/mm^2$	Aceros con $f_y = 500\ N/mm^2$
Pilares	4,0	4,0
Losas[1]	2,0	$\boxed{1,8}$

[1] Cuantía mínima de cada una de las armaduras, longitudinal y transversal repartida en las dos caras. Para losas de cimentación y zapatas armadas, se adoptará la mitad de estos valores en cada dirección dispuestos en la cara inferior.

In this table we find that the minimum geometric amount in $_{0/00}$(part per milliard) for a footing is a half of the indicated value, which is $0.9_{0/00}$. So, applying this value to the equation of the geometric amount I have:

$$\rho = \frac{A_s}{A_c}$$

Where $A_c = a \cdot d = 300 \cdot 57 = 17100\ cm^2$ is the useful area of the footing.

$$A_s = \frac{0.9}{1000} \cdot 17100 = 15.39\ cm^2$$

As this result is smaller than the other previously calculated I will use it to define the reinforcement of the footing, because it is the most restrictive value.
So, $A_s = 15.39\ cm^2$.

$$Area\ of\ a\ steel\ bar\ of\ 12\ mm\ diameter = \pi \cdot \frac{diameter^2}{4} = \pi \cdot \frac{1.2^2}{4} = 1.13\ cm^2$$

$$Distance\ between\ bars = 15\ cm \rightarrow \frac{a}{15} = \frac{300}{15} = 20\ steel\ bars$$

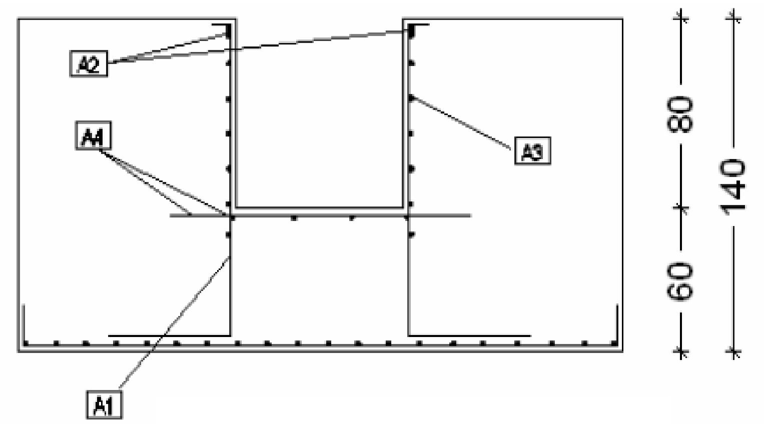
$$Total\ area\ of\ steel\ bars = 20 \cdot 1.13 = 22.6\ cm^2$$

$$Area\ of\ steel\ bars \geq A_s \rightarrow 22.6 \geq 15.39\ cm^2 \rightarrow Verified$$

After performing these operations I can say that what I need is to place in the footing a mesh of $15x15\ cm$ compounded by steel bars of $12\ mm$ diameter.

8.3.7 Reinforcement of the calyx or cup

For the reinforcement of the calyx of the holes I will use a standard used by Tecnyconta for this kind of columns, which would be the next one:



A1: 6 $vertical\ bars\ of\ diameter$ 20 $mm\ per\ side$

A2: 4 $upper\ perimetral\ fences\ of\ diameter$ 16 mm

A3: 6 $bottom\ perimetral\ fences\ of\ diameter$ 10 mm

A4: 4 $local\ reinforcements\ under\ the\ column, of\ diameter$ 12 $mm\ in\ both\ directions$

8.4 Calculation of the attachment beams or foundation strings

I could consider the sill as the attachment beam, and it would be correctly done in terms of structural design. However, I need a beam to support the enclosures which will be standing on the floor.

In order to design this attachment beam y will consider the beam as a double clamped beam.

As below this beam I will have 5 cm of levelling concrete, I won´t have to meet the specifications of EHE-08 point 3.7.2.4.1, which says that we need a covering of 7 cm for elements concreted against the terrain.

8.4.1 Pre-dimensioning of the beam

First of all, as I did with the footings, I will pre-dimension the beam, taking into account that if I don´t want to check the buckling the beam will have to meet the following formula:

$$b \geq \frac{l}{20} = \frac{5}{20} = 0.25\ m$$

Where:
l → Maximum length between the surface of the adjacent footing.

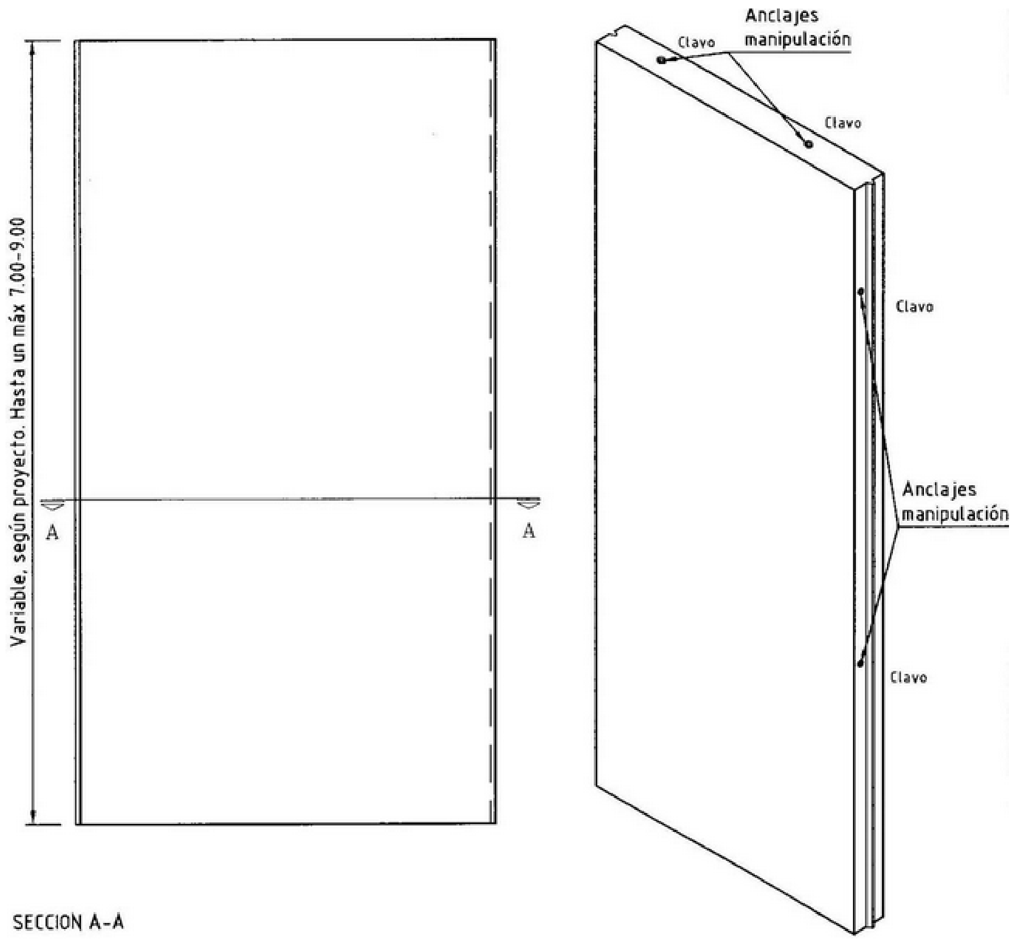
b → Smallest size of any of the dimensions of the transversal area of the beam.

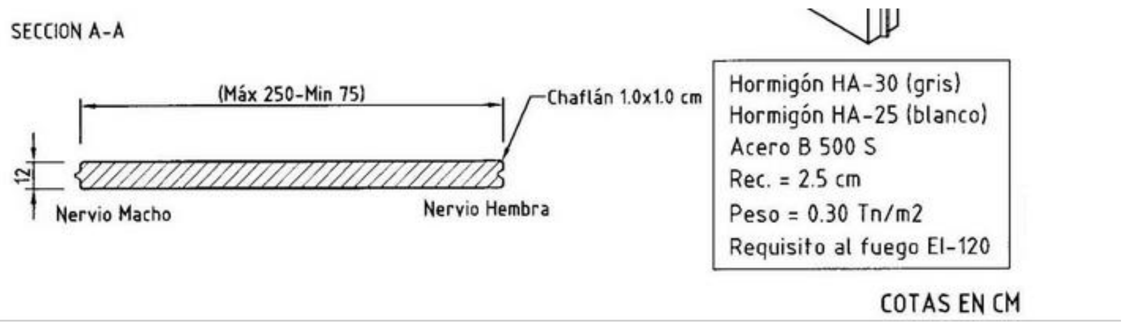
Taking into account the previous condition, I will define my beam as:

$$a_{beam} = 0.35\ m \geq 0.25\ m$$
$$b_{beam} = 0.35\ m \geq 0.25m$$

8.4.2 Actions to consider for the calculation of the beam

First of all I have to define the panels for the enclosure that I will be using:





Now, I can calculate the actions:

- Self-weight of the beam:
  Taking into account that the specific weight of concrete is $\gamma_h = 2500\ kg/m^3$ and the dimensions of the beam:
  $$P_{beam} = \gamma_h \cdot Area = 2500 \cdot 0.35 \cdot 0.35 = 306.25 \frac{kg}{m} = 3.06\ kN/m$$
- Self-weight of the panels of the enclosure:
  Taking into account that the panels are $l_{panel} = 2.5\ m$ length, and that its self-weight is $s - w_{panel} = 300\ kg/m^2$:
  $$P_{panel} = s - w_{panel} \cdot h_{panel} = 300 * 7.45 = 2235 \frac{kg}{m} = 22.35\ kN/m$$

Adding these two values and applying the enlargement coefficient from table 4.1 of the Basic Document SE, the same as in some preliminary steps and which says that for self-weight in permanent actions its value is $\gamma_{s-w} = 1.35$, I have the linear load:

$$q_{linear} = \left(P_{beam} + P_{panel}\right) \cdot \gamma_{s-w} = (3.06 + 22.35) \cdot 1.35 = 34.31\ kN/m$$

So, the value of the actions will be:

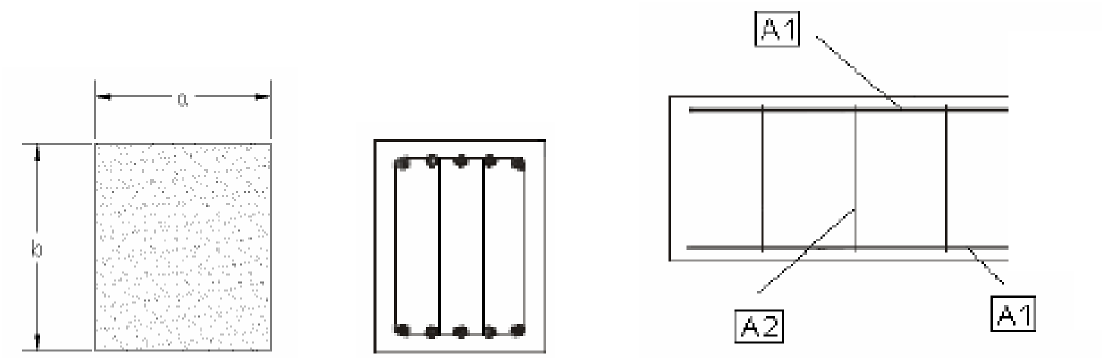
- $M_d = \frac{q_{linear} \cdot l^2}{12} = \frac{33.21 \cdot 1 \cdot 5^2}{12} = 71.47\ kN \cdot m$

- $V_d = \frac{q_{linear} \cdot l}{2} = \frac{33.21 \cdot 1 \cdot 5}{2} = 85.77\ kN$

8.4.3 Checking of the reinforcement of the attachment beam

The first step to accomplish when calculating the reinforcement of the attachment beam is to pre-dimension it, by performing after that some checking of the limit states that will tell us if the chosen dimensioning is enough to stand the previous actions.

I will consider that the attachment beam will have a covering of 3 cm and that it will be the following one, given by company:



- A1: 5 longitudinal bars of 20 mm diameter per side
- A2: Stirrups of 4 bars of 8 mm each 20 cm

Other values of interest:

$M_u = 265\ kN \cdot m;$ $\quad V_{u1} = 350\ kN;$ $\quad V_{u2} = 236\ kN;$ $\quad W_k = 0.2\ mm$

In order to verify the longitudinal and transversal reinforcement for the beam is enough for the applied actions, I will have to compare them with the values obtained for this beam. This values are:

- $M_u = 265\ kN \cdot m \rightarrow$ Ultimate bending moment stand.
- $V_{u1} = 350\ kN \rightarrow$ Ultimate shearing to compression for the web.
- $V_{u2} = 236\ kN \rightarrow$ Ultimate shearing to traction for the web.
- $W_k = 0.2\ mm \rightarrow$ Opening of the crack.

With this values I can check that:

$$M_d = 71.47\ kN \cdot m \leq M_u = 265\ kN \cdot m \rightarrow Verified$$

8.4.3.1 Verification of the applied shear (ULS)" V_d"

In order to be able to consider the effect of the transversal reinforcement considered in the attachment beam for the verification of the ultimate shear stand, as indicated in point 44.2.3.1 of the EHE-08, I will have to check the following conditions for the distance between transversal reinforcement "s_t":

$s_t \leqslant 0.60 \cdot d \cdot (1 + cotg\alpha)$ and $s_t \leqslant 450\ mm$

If $\frac{1}{5} \cdot V_{ul} < V_d \leqslant \frac{2}{3} \cdot V_{ul}$

Where:

$V_d = 85.77\ kN;$ $V_{u1} = 350\ kN;$ $d = 340\ mm;$

$\alpha = 90°;$ $s_t = 200\ mm.$

$$\frac{1}{5} \cdot V_{u1} < V_d \leqslant \frac{2}{3} \cdot V_{u1} \rightarrow \frac{350}{5} \leq 83.03 \leq 2 \cdot \frac{350}{3} \rightarrow 70 \leq 85.77 \leq 233.33\ kN \rightarrow Verified$$

$$s_t \leqslant 0.60 \cdot d \cdot (1 + cotg\alpha) \rightarrow s_t \leqslant 204\ mm \rightarrow Verified$$

$$s_t \leqslant 450\ mm \rightarrow Verified$$

So, I can consider the shear reinforcement, as the distance is smaller than the one required.

Taking into account point 44.2.3 of the EHE-08, for a shear reinforcement it is necessary to verify:

$$V_d \leqslant V_{u1}\ and\ V_d \leqslant V_{u2}$$

$$V_d = 85.77 \leq 236 \leq 350\ kN \rightarrow Verified$$

8.4.3.2 Verification of the cracking bending moment due to wind action (ULS) "M_f"

According to table 8.2.2 from EHE-08, the attachment beam is subjected to a general class of exposition of type IIa, as we can see here:

CLASE GENERAL DE EXPOSICIÓN				DESCRIPCIÓN	EJEMPLOS
Clase	Subclase	Designación	Tipo de proceso		
No agresiva		I	Ninguno	– Interiores de edificios, no sometidos a condensaciones. – Elementos de hormigón en masa.	– Elementos estructurales de edificios, incluido los forjados, que estén protegidos de la intemperie.
Normal	Humedad alta	IIa	Corrosión de origen diferente de los cloruros	– Interiores sometidos a humedades relativas medias altas (> 65%) o a condensaciones. – Exteriores en ausencia de cloruros, y expuestos a lluvia en zonas con precipitación media anual superior a 600 mm. – Elementos enterrados o sumergidos.	– Elementos estructurales en sótanos no ventilados. – Cimentaciones. – Estribos, pilas y tableros de puentes en zonas, sin impermeabilizar con precipitación media anual superior a 600 mm. – Tableros de puentes impermeabilizados, en zonas con sales de deshielo y precipitación media anual superior a 600 mm. – Elementos de hormigón, que se encuentren a la intemperie o en las cubiertas de edificios en zonas con precipitación media anual superior a 600 mm. – Forjados en cámara sanitaria, o en interiores en cocinas y baños, o en cubierta no protegida.

According to table 5.1.1.2 of the EHE-08, for a reinforced element with a general class of exposition IIa, the maximum width admitted "W_{max}" is $0.3\ mm$. So:

$$W_k \leq W_{max} \rightarrow 0.2 \leq 0.3\ mm$$

Which means that:

$$M_f \leq M_a \leq M_{f0.3}\ kN \cdot m$$

This is, I verify that there exists a cracked zone, and that the element stands the cracking bending moment correctly, as the width of the opening is smaller than the one allowed.

8.4.3.3 Verification of the minimum geometric amount for the principal reinforcement

In previous steps I have verified that the pre-dimensioning for the reinforcement of the attachment beam is enough to stand the actions acting on it. So, in order to say that this reinforcement is correct, I will have to verify that it meets the indications of the EHE-08 for the minimum amount required for a reinforcement.

In table 42.3.5 of the EHE:

Tipo de elemento estructural	Tipo de acero	
	Aceros con $f_y = 400\ N/mm^2$	Aceros con $f_y = 500\ N/mm^2$
Vigas[4]	3,3	2,8

Which says that the minimum geometric amount for a beam is $2.8_{0/00}$. So, applying this value for the equation of the amount I have:

$$\rho = \frac{A_s}{A_c}$$

Where $A_c = a \cdot d = 35 \cdot 34 = 1190\ cm^2$ is the useful area of the footing.

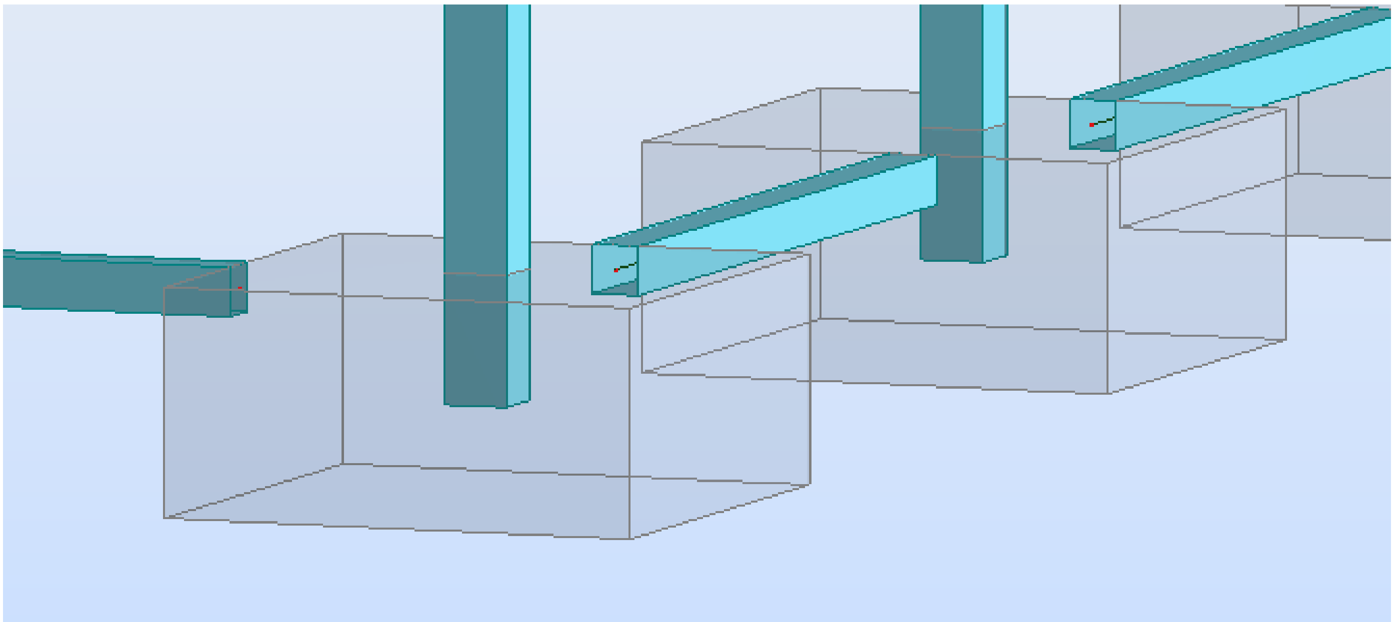
$$A_s = \frac{2.8}{1000} \cdot 1190 = 3.33\ cm^2$$

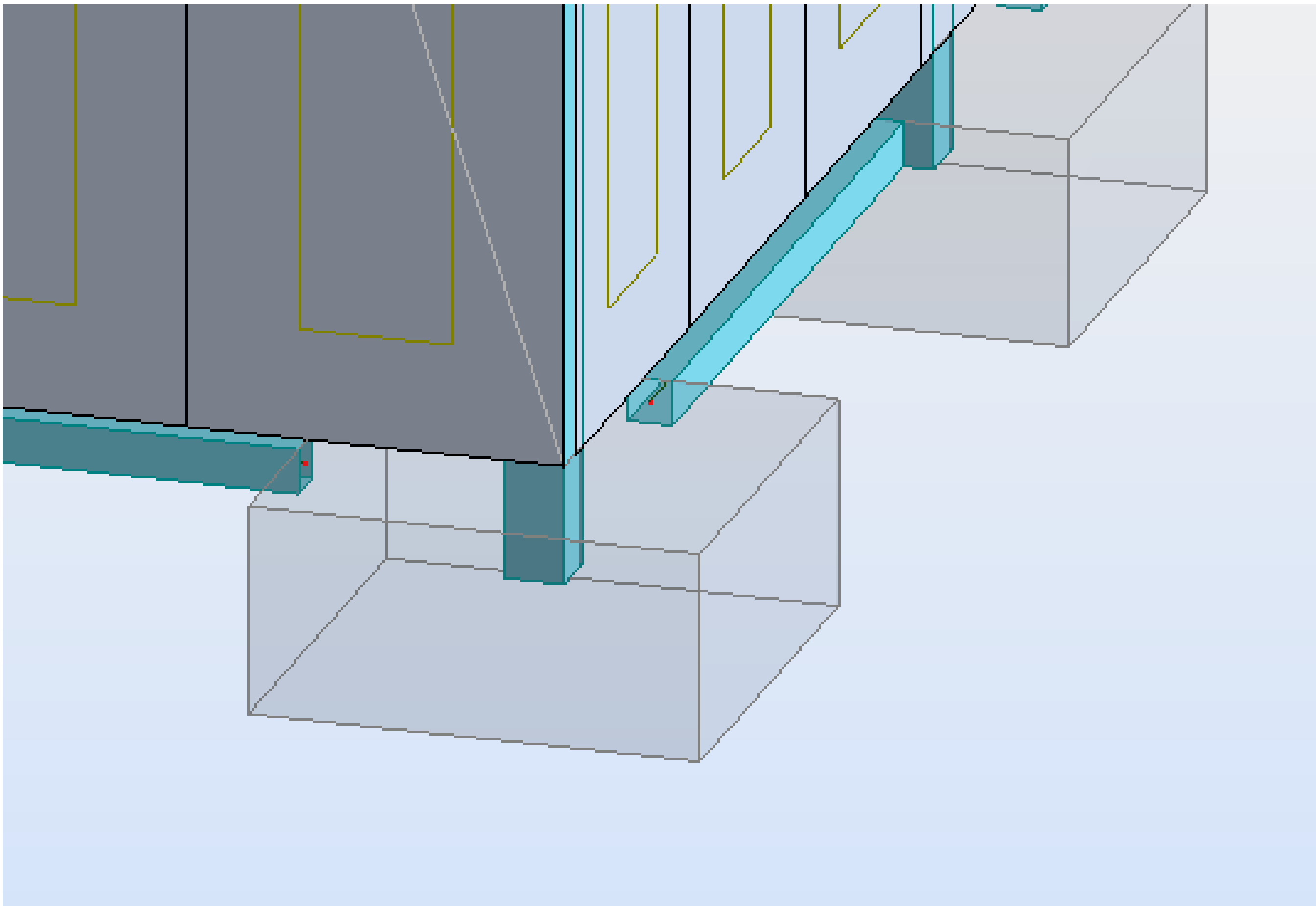
The area of the passive reinforcement acting for traction "A_s" considered for the top or bottom surface of the attachment beam is:

$$A_s = \frac{\pi \cdot diameter^2}{4} \cdot n^{\circ} of\ bars = \frac{\pi \cdot 2^2}{4} \cdot 5 = 15.71\ cm^2$$

As the considered area of the reinforcement is larger than the one required by EHE-08 for the minimum geometric amount, I can say that the reinforcement considered is correct for the development of the attachment beams for the foundation.

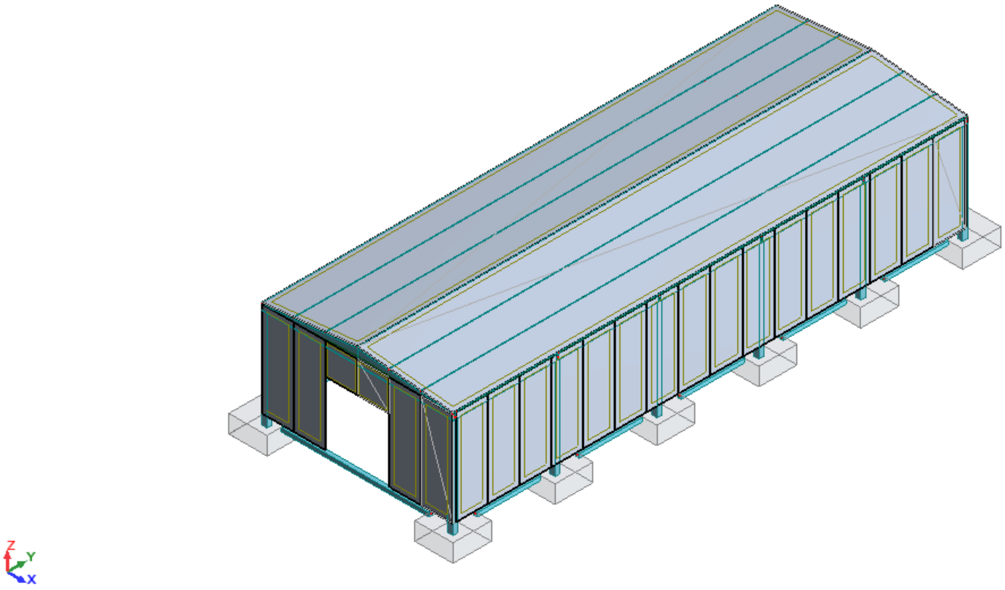
8.5 Final disposition of the footings and attachment beams



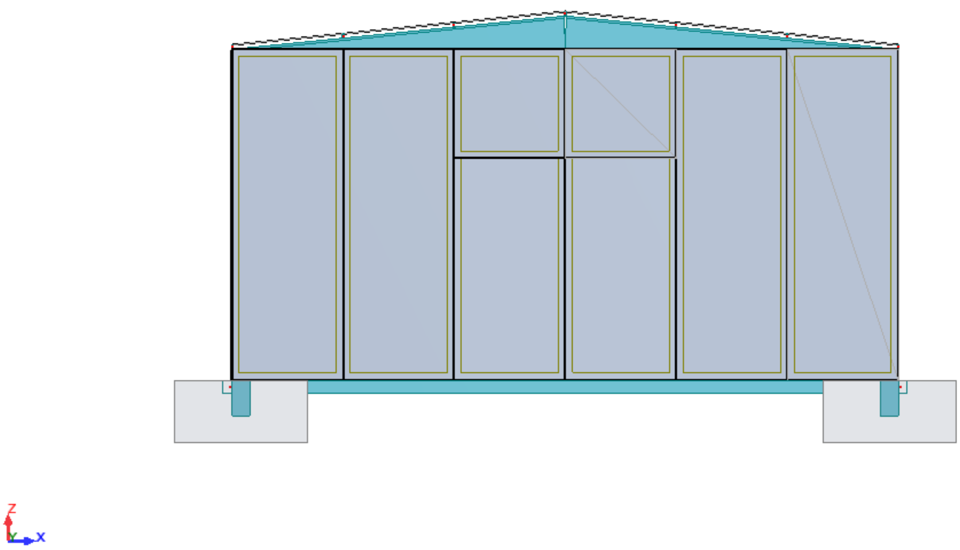


9 VIEWS OF THE 3D STRUCTURE

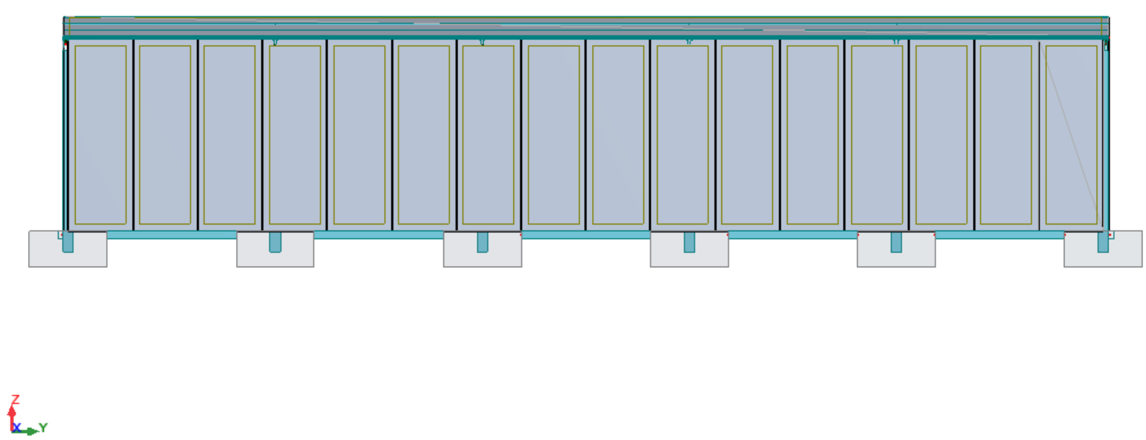
General view:



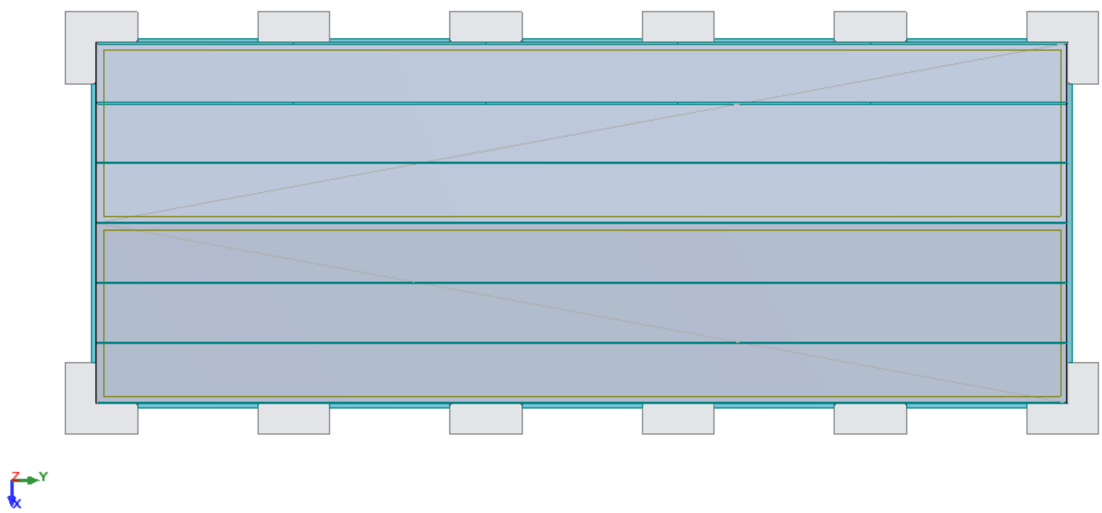
Front view:



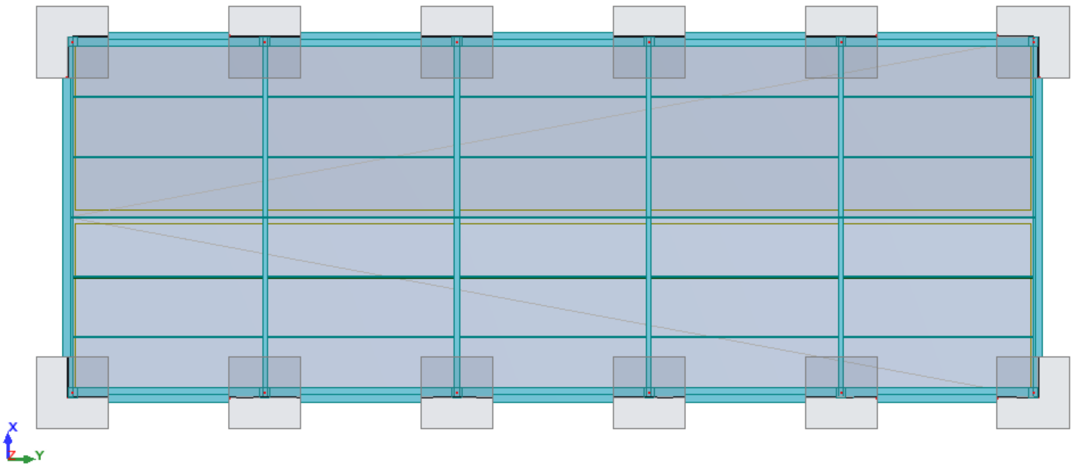
Side view



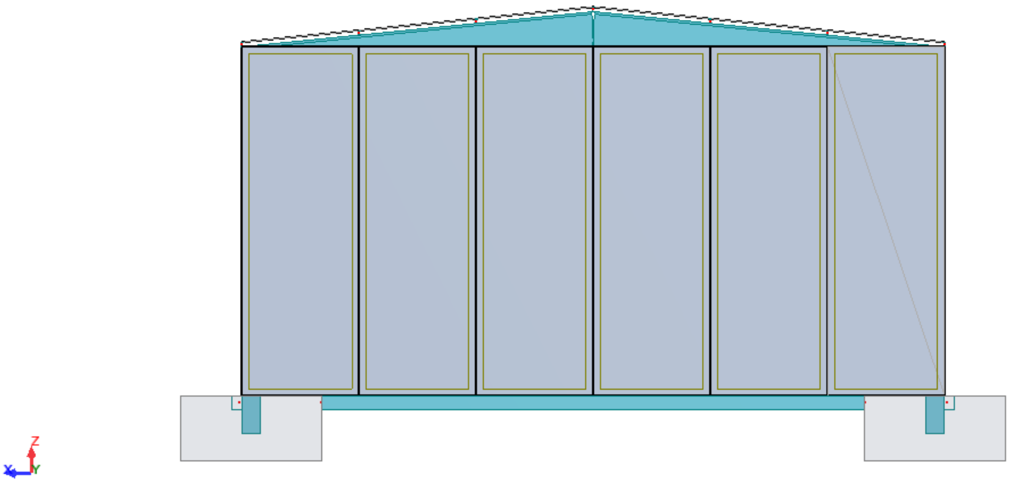
Top view:



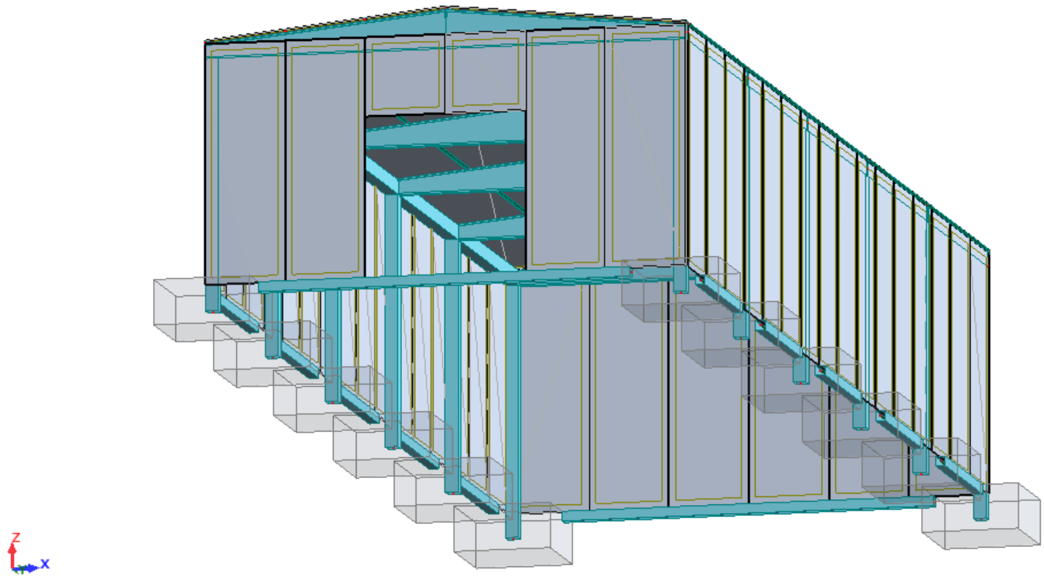
Bottom view:



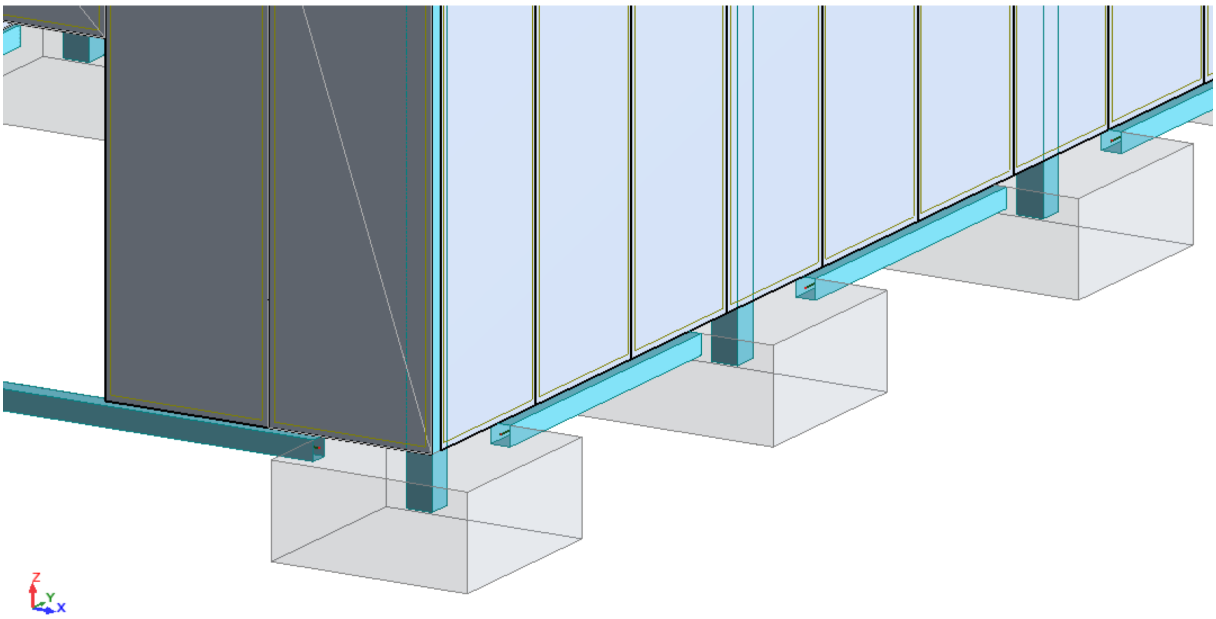
Rear view



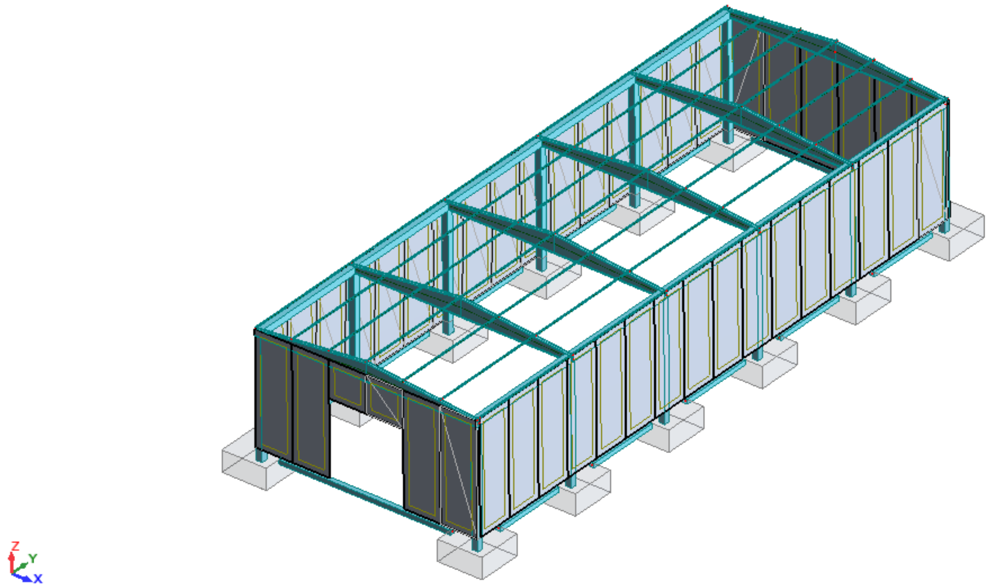
Bottom-Front view:



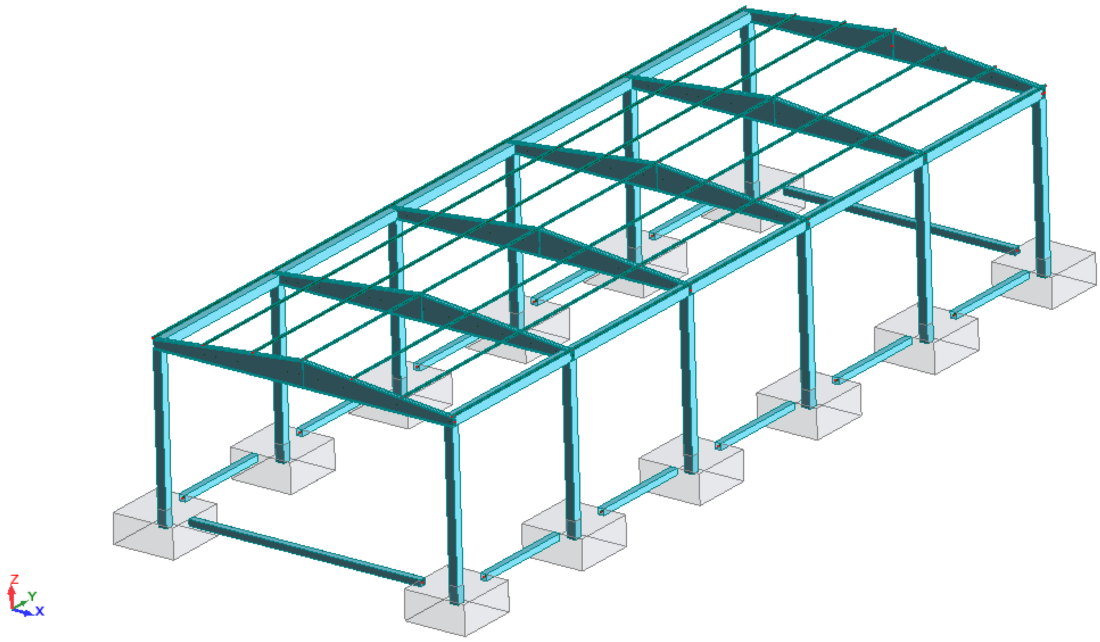
Foundations and attachment beams:



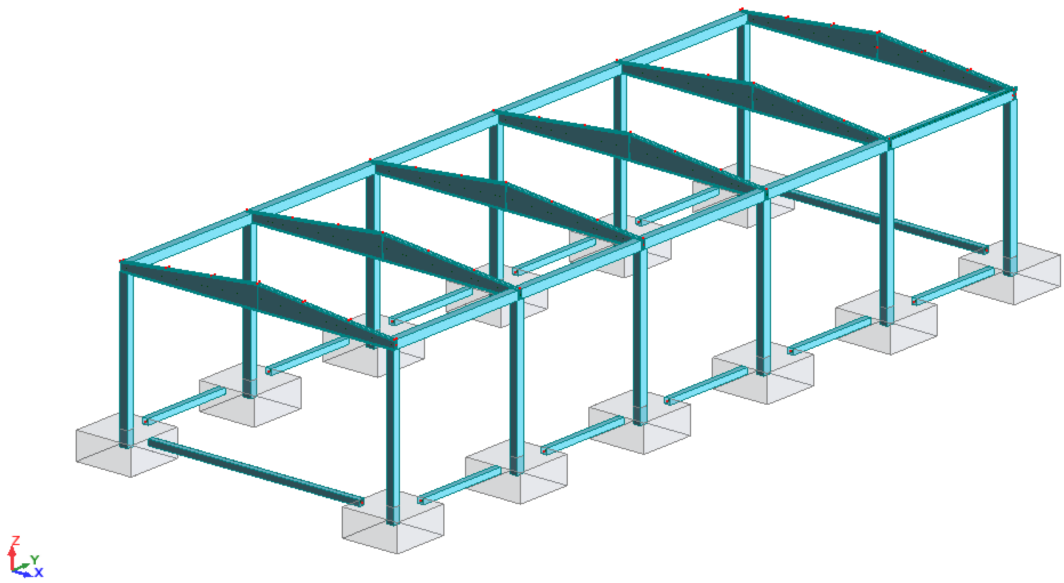
View without roof:



View without roof and enclosures:



Structural view:



10 VIEWS WITH DIMENSIONS

11 CONCLUSIONS

According to the previous calculations I can conclude that a building with the specified dimensions would stand the actions applied on it and would be structurally safe.

Panels of the entrance will be lighter than the concrete panels of the enclosures. For this reason, it wasn´t necessary to perform any different calculation for them. I have considered the entrance as $5\ m\ x\ 5\ m$ in the front wall.

As we can see in the pictures, attachment beams are not completely centred. This is because their main function is to support the heavy enclosures. As they are not needed for their structural response, this disposition is correct, centred in the axis of the enclosure panels.

12 BIBLIOGRAPHY

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- *Código Técnico de la Edificación (CTE).* Mainly:
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    - *Documento Básico – Seguridad Estructural Acciones en la Edificación*
- Notes:
    - *Fundamentals of Structural Design*
    - *Elasticidad y Resistencia de Materiales I and II*
    - *Ingeniería Estructural*
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- *www.becam.com*
- *www.tecnyconta.com*
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- *www.lufort.es*
- *www.forte.es*

13 PROGRAMS USED

- Autodesk AutoCAD 2015
- Autodesk Robot Structural Analysis Professional 2014

RESUMEN

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Fecha de lectura: 17 de Junio de 2014

Calificación: 10 (Según criterio UC3M: 10/10)
5 (Según criterio POLSL: 5/5)

1. INTRODUCCIÓN

1.1. Introducción a la idea

Mi principal motivación ha sido la de incorporar conocimientos relativos al diseño de naves industriales, ya que mi meta es llegar a desarrollar proyectos en ese campo a nivel laboral.

1.2. Idea preliminar de una nave industrial

Una nave industrial es un edificio diseñado para la producción y/o almacenamiento de bienes.

Principalmente de tres tipos:

- Hormigón: Barata.
- Acero: Rápida, pero más cara.
- Mixta: Elementos de acero y elementos de hormigón.

2. EDIFICIO

2.1. Objetivo del proyecto

Análisis de la respuesta de una nave industrial mixta sometida a diferentes tipos de cargas.

2.2. Datos iniciales de la construcción

2.2.1. Material

El material utilizado será principalmente hormigón prefabricado, que será usado para todo excepto para la cubierta y las correas de la cubierta.

2.2.2. Dimensiones y forma

La nave tendrá una planta rectangular, y las medidas serán 15 metros de ancho, 40.4 metros de largo y 8.2 metros de alto.

La cubierta tendrá una inclinación del 10%. Las zapatas serán diseñadas una vez se hayan calculado las fuerzas ejercidas sobre ellas.

3. REQUISITOS DEL TERRENO

De acuerdo a las tablas 3.1 y 3.2 del Documento Básico SE- C, el edificio quedará clasificado como Tipo C-1, y el terreno escogido será del Grupo T-2.

4. CARGAS VARIABLES EN LA CUBIERTA

4.1. Carga de nieve

4.1.1. Determinación de la carga de nieve “q_n”

Su determinación se realizará a través del Documento Básico SE-AE, mediante la fórmula: $q_n = \mu \cdot s_k$

4.1.2. Valor característico de la carga de nieve “s_k”

El valor característico se obtiene de la tabla 3.8 del Documento Básico SE-AE. Al estar localizada en Madrid, este valor será: $s_k = 0.6\ kN/m^2$

4.1.3. Coeficiente de forma de la cubierta "μ"

El coeficiente de forma de la cubierta depende de su inclinación. Dado que en mi caso, esta inclinación es menor que 30 grados (10% = 5.7 grados), este valor será: $\mu = 1$

4.1.4. Valor final de "q_n"

Introduciendo todos los valores en la fórmula inicial:

$$q_n = \mu \cdot s_k = 1 \cdot 0.6 = 0.6 \frac{kN}{m^2} = 60 \frac{kg}{m^2}$$

4.2. Carga de viento

4.2.1. Determinación de la acción del viento en la cubierta "q_e"

Para la obtención del valor de la acción del viento sobre la cubierta utilizaremos la fórmula:

$$q_e = q_b \cdot c_e \cdot c_p$$

4.2.2. Determinación de la presión dinámica del viento "q_b"

La presión dinámica del viento depende de la ubicación geográfica de la edificación. Dado que, tras su cálculo, el valor obtenido es menor que el valor promedio aceptable para toda España, utilizaré este último para un mayor margen de seguridad: q_b =0.5kN/m^2

4.2.3. Coeficiente de exposición "c_e"

El coeficiente de exposición se obtiene a través de la tabla 3.4 del Documento Básico SE-AE. Dado que en esta tabla no cuenta con un valor determinado para la altura de mi edificio (8.2 metros), deberé interpolar. El valor final obtenido es:

c_e = 1.62.

4.2.4. Coeficiente eólico o de presión "c_p"

El coeficiente de presión depende de la dirección relativa del viento, la forma de la cubierta, la posición del elemento considerado y del área de influencia. Para obtenerlo, utilizaré la tabla D.6 del Documento Básico SE-AE. Mediante una interpolación para el ángulo de mi cubierta, el valor obtenido es: $c_p = 0.186$

4.2.5. Valor final de "q_e"

Introduciendo todos los valores anteriores, el valor final para la acción de viento es: $q_e = q_b \cdot c_e \cdot c_p = 0.5 \cdot 1.62 \cdot 0.186 = 0.151 \frac{kN}{m^2} = 15.1 \frac{kg}{m^2}$

5. CORREAS DE LA CUBIERTA

5.1. Tipo de correa

Imágenes en el proyecto completo

5.2. Tipo de cubierta metálica para la cubierta

Imágenes en el proyecto completo

5.3. Determinación de los estados límite

Para los estados límite tendré en cuenta la acción de la nieve, el viento y el peso propio de correas y cubierta. El comportamiento será aceptable si el valor de las acciones es menor que el valor resistido por las correas: $E_d \leqslant R_d$

5.4. Combinación de acciones

Los efectos de las acciones aplicadas en las correas se determinan mediante la combinación de acciones e influencias simultaneas. Consideramos dos tipos:

5.4.1. Estado Límite de Servicio:

$$\sum_{j\geq 1} G_{k,j} + P + Q_{k,1} + \sum_{i>1} \Psi_{0,i} \cdot Q_{k,i}$$

5.4.2. Estado Límite Último:

$$\sum_{j\geq 1} \gamma_{G,j} \cdot G_{k,j} + \gamma_P \cdot P + \gamma_{Q,1} \cdot Q_{k,1} + \sum_{i>1} \gamma_{Q,i} \cdot \Psi_{0,i} \cdot Q_{k,i}$$

Tendremos que chequear que:

$M_f = 20.50\ kN \cdot m \geq \frac{l^2 \cdot \sum q_i}{8}$ → Momento flector para la fisura de las correas

$M_u = 31.20\ kN \cdot m \geq \frac{l^2 \cdot \sum q_i \cdot \gamma_i}{8}$ → Momento flector ultimo soportado por las correas

$V_u = 25.60\ kN \geq \frac{l \cdot \sum q_i \cdot \gamma_i}{2}$ → Cortante último soportado por las correas

Donde:

$$q_i = q_{peso\ propio\ correas} + q_{cubierta} \cdot s_i + q_n \cdot s_i + q_e \cdot s_i$$

$$l = l_{total} - \frac{l_{apoyo\ 1} + l_{apoyo\ 2}}{2}$$

5.5. Verificación del Estado Limite de Servicio (SLS)

Comprobaré:

$M_f = 20.50\ kN \cdot m \geq \frac{l^2 \cdot \sum q_i}{8} =$, tomando Ψ_0 = 0.5 para nieve y 0.6 para viento. De esta forma se obtienen los valores para distancia máxima entre correas:

$s_1{}' = 3.54\ m$ y $s_2{}' = 5.27\ m$

5.6. Verificación del Estado Límite Último (ELU)

En este caso comprobaré:

$M_u = 31.20\ kN \cdot m \geq \frac{l^2 \cdot \sum q_i \cdot \gamma_i}{8}$ y $V_u = 25.60\ kN \geq \frac{l \cdot \sum q_i \cdot \gamma_i}{2}$

Los valores obtenidos son:
$s_1 = 3.62\ m\ ;\ s_2 = 5.40\ m\ ;\quad s_3 = 5.89\ m\ ;\quad s_4 = 8.78\ m$
De todos estos valores, tomamos el menor como límite para la disposición de las correas, cuyo valor es $s_1{}' = 3.54\ m$. Como la distancia total a cubrir es de $7.53\ m$, situaré las correas equidistantes a $s_{final} = \frac{7.53}{3} = 2.51\ m$.

5.7. Disposición final de las correas

Imágenes en el proyecto completo

6. <u>CÁLCULO DE LAS VIGAS DELTA</u>

6.1. Vigas delta

Ficha técnica en el proyecto completo

6.1.1. Verificación del Estado Límite Último (ELU)

Para el chequeo del ELUS comprobaré el momento flector aplicado y el esfuerzo cortante aplicado. El valor de las acciones deberá ser menor que límite de la viga:

$$S_d \leq R_d$$

6.1.1.1. Verificación del momento flector aplicado "$\mathrm{M_d}$"

Para el momento flector aplicado, comprobaré que $M_d \leq M_u$, siendo $M_d = \sum \frac{q_i \cdot l^2}{8} \cdot \gamma_i$ y M_u el límite de la viga.

Para las cargas del momento, tendré en cuenta el peso propio de la viga, de los paneles de la cubierta y la carga de nieve.

Tras realizar los cálculos oportunos:

$M_d = M_{peso\ propio} + M_{cubierta} + M_n = 135.47 + 12.32 + 191.84 = 339.62\ kN \cdot m \leqslant 850\ kN \cdot m$, quedando verificado.

6.1.1.2. Verificación del cortante aplicado "$\mathrm{V_d}$"

Las condiciones para considerar la armadura a cortante de la viga se obtienen del artículo 44.2.3.4.1 de la EHE-08. Tras su cálculo, vemos que se puede considerar el refuerzo para cortante.

Por lo tanto, verificaremos que:

$V_d \leqslant V_{u1}$ y $V_d \leqslant V_{u2}$, siendo $V_d = V_{peso\ propio} + V_{cubierta} + V_n$

Realizando los cálculos apropiados, tenemos que:

$V_d = 37.11 + 3.37 + 52.56 = 93.05\ kN \leqslant 284.7\ kN \leqslant 710\ kN$, quedando verificado.

6.1.2. Verificación del Estado Límite de Servicio (ELS)

Para el chequeo del ELS comprobare el momento flector para fisura y el desplazamiento máximo admisible. El efecto de las acciones deberá ser menor que el valor admisible para la sección:

$$E_d \leq C_d$$

6.1.2.1. Verificación del momento flector para fisura “M_f”

Primero obtenemos la apertura máxima para el elemento. En este caso es $W_{max} = 0.3\ mm$. Con ello sabemos que deberemos tener en cuenta el momento máximo en caso de fisura.

Hallando el momento aplicado sin coeficiente de seguridad, sabemos si existirá dicha fisura o no:

$$M_d = M_a = \sum \frac{q_i \cdot l^2}{8} = M_{peso\ propio} + M_{cubierta} + M_n$$

Realizando los cálculos pertinentes:

$M_d = 100.35 + 9.12 + 127.90 = 237.36\ kN \cdot m \leqslant 708.2\ kN \cdot m = \mathrm{M_f}$, por lo que no hay fisura: $M_a \leq M_f \leq M_{f0.3}$

6.1.2.2. Verificación de la deformación máxima admisible “$\Delta \mathrm{fl}_{\mathrm{total}}$”

Para dicha verificación, consideraré unos desplazamientos máximos en la viga de $0.0\ mm$ a la llegada, $8.0\ mm$ tras aplicar las cargas de la cubierta, y $15.0\ mm$ cuando el tiempo tienda a infinito.

Mediante el artículo 50 de la EHE-08 verificamos que $fl_{total} \leq \Delta fl_{total}$.

Obteniendo $\Delta fl_{total} = 39.2\ mm$ se comprueba que no supera el valor máximo, por lo tanto el diseño es correcto.

6.1.3. Disposición final de las vigas delta

Imágenes en el proyecto completo.

7. <u>COLUMNAS</u>

7.1. Introducción

Para calcular las columnas, primero deberé obtener el valor de las acciones aplicadas sobre ellas. Como el peso propio de la nieve es una carga estable, solo calcularé la acción del viento.

7.2. Acción del viento en las columnas

Se realiza mediante la misma ecuación explicada anteriormente para la cubierta.

7.2.1. Coeficiente de exposición “c_e”

Mediante las fórmulas: $F = k \cdot \ln\left(\frac{\max(z,Z)}{L}\right)$ y $c_e = F \cdot (F + 7k)$

Los valores de los parámetros se obtienen de la tabla D.2 del Documento Básico SE-AE. De esta forma, obtenemos el valor del coeficiente de exposición dependiendo de la altura, tal y como viene en la tabla del proyecto completo.

7.2.2. Coeficiente eólico o de presión “c_p”

El coeficiente de presión se obtiene mediante la tabla D.3 del Documento Básico SE-AE, para dos direcciones visiblemente ortogonales para ambos sentidos. De este modo, obtenemos la tabla mostrada en el proyecto completo. Una vez tenemos este valor, solo hay que calcular los valores de la acción del viento para las cuatro direcciones y las distintas alturas. Incluido en el proyecto completo.

7.3. Cálculo de las columnas

7.3.1. Tipo de columna

Ficha técnica en el proyecto completo.

7.3.2. Cargas consideradas

Las cargas consideradas han sido el peso propio de la cubierta y de las correas, la acción de la nieve, el peso propio del canal de hormigón y el peso propio de las vigas delta.

7.3.3. Hipótesis consideradas

He considerado 13 hipótesis diferentes, jugando con las cargas permanentes, las sobrecargas y la acción del viento, para obtener la peor respuesta posible y realizar el diseño con ella.

7.3.4. Numeración de las columnas

Imágenes en el proyecto completo

7.3.5. Casos a considerar

7.3.5.1. Columnas de las esquinas

Tendré en cuenta las cargas indicadas anteriormente. Para estas columnas, la distancia considerada para la acción del viento será la mitad que en las interiores. Los resultados con los valores obtenidos se pueden encontrar en el proyecto completo, una tabla por columna.

7.3.5.2. Columnas interiores

Tendré en cuenta las cargas indicadas anteriormente. Para estas columnas, la distancia considerada para la acción del viento será el doble que en las esquinas. Los resultados con los valores obtenidos se pueden encontrar en el proyecto completo, una tabla por columna.

El peor resultado se obtuvo para las columnas 8, 9, 10 y 11 con la hipótesis 13, así que serán estos valores los utilizados para las verificaciones.

7.3.6. Verificación del Estado Límite Último(ELU)

Para el chequeo del ELUS comprobaré el momento flector aplicado y el esfuerzo cortante aplicado. El valor de las acciones deberá ser menor que límite de la viga:

$$S_d \leq R_d$$

Como coeficiente de seguridad aplicaré un valor intermedio entre las cargas permanentes y variables, esto es: $\gamma_{aprox} = 1.42$

7.3.6.1. Verificación del momento flector aplicado "$\mathrm{M_d}$"

La condición a satisfacer será: $M_d \leq M_u$.

Dado que el valor obtenido del cálculo era $M_a = 263.22\ kN \cdot m$, aplicando el coeficiente de seguridad tenemos: $M_d = M_a \cdot \gamma_{aprox} = 263.22 \cdot 1.42 = 373.78\ kN \cdot m$.

Y por lo tanto, se cumple que $M_d = 373.78\ kN \cdot m \leq M_u = 590\ kN \cdot m$

7.3.6.2. Verificación del cortante aplicado "V_d"

Para comprobar si se puede considerar la armadura a cortante, comprobaremos condiciones similares al caso de la viga delta. Una vez realizadas dichas comprobaciones, vemos que se puede considerar. Por lo tanto, deberemos verificar que: $V_d \leqslant V_{u1}$ y $V_d \leqslant V_{u2}$. $V_d = 68.39 \leq 334 \leq 1440\ kN$, quedando verificado.

7.3.7. Verificación del Estado Límite de Servicio (ELS)

Para la comprobación del ELS comprobaré el momento flector para fisura y el desplazamiento horizontal en dirección Y. El valor de las acciones no deberá superar el valor límite admisible: $E_d \leq C_d$

7.3.7.1. Verificación del momento flector para fisura "M_f"

El valor de la fisura según la ficha técnica para la carga aplicada es de $W_k = 0.38\ mm$. Según las tablas 8.2.2 y 5.1.1.2 de la EHE-08, la apertura máxima para la fisura es de $W_{max} = 0.4\ mm$

Se cumple la regla $W_k \leq W_{max} \rightarrow 0.38 \leq 0.4\ mm \rightarrow M_f \leq M_a \leq M_{f0.4}\ kN \cdot m$. Hay fisura pero la columna aguanta.

7.3.7.2. Verificación del desplazamiento horizontal "Δd_y" en dirección Y

El desplazamiento horizontal es de $\Delta d_y = 15\ mm$.

Según el apartado 4.3.3.2 del Documento Básico SE, hay fallo si:

$\Delta d_y \leq \frac{1}{500} \cdot h_{element}$.

Por lo tanto: $\Delta d_y \leq 1/500 \cdot h_{element} \rightarrow 15 \leq \frac{1}{500} \cdot 7.8 \rightarrow 15 \leq 15.6\ mm.$

8. CÁLCULO DE LAS CIMENTACIONES DE LA NAVE INDUSTRIAL

8.1. Tipo de zapata

La cimentación de la nave se realizará mediante zapatas aisladas rígidas, con unión en cáliz.

Ficha técnica en el proyecto completo (Propiedades del material).

8.2. Recomendaciones generales para cimentaciones

Para los cálculos de las cimentaciones se seguirán las indicaciones dadas en el Documento Básico SE-C.

8.3. Cálculo de las zapatas

8.3.1. Acciones consideradas y pre-dimensionamiento de la zapata

Las acciones a considerar serán las más desfavorables, es decir, hipótesis 13 para las columnas mencionadas anteriormente. Haremos un dimensionado estimado y veremos si cumple las condiciones necesarias. Las medidas serán:

$a = 3\ m$; $b = 3\ m$; $h = 1.40\ m$

8.3.2. Comprobación del vuelco

Comprobamos que los momentos estabilizadores son mayores que los momentos que tienden al vuelco: $M_1 \geq M_2 \rightarrow (N_a + P) \cdot \frac{a}{2} \geq (M_a + V_a \cdot h) \cdot \gamma_E$.

Realizando los cálculos necesarios tenemos que:

$M_1 \geq M_2 \rightarrow 637.53 \geq 595.18\ kN \cdot m$, por lo que soporta el vueco.

8.3.3. Comprobación de la condición de zapata rígida

La condición para poder considerar la zapata como zapata rígida es la siguiente: $Distancia < 2 \cdot h$, siendo Distancia la distancia entre el extreme de la Zapata y el comienzo de la columna. En este caso: $Distancia < 2 \cdot h \rightarrow 130 < 280\ cm$, por lo que queda verificado.

8.3.4. Comprobación del deslizamiento

Dado que incluiré vigas de atado entre las zapatas para soportar los cerramientos, esta condición se satisface directamente.

8.3.5. Comprobación del colapso del terreno o de la deformación admisible del terreno "$\sigma_{terrain}$"

Primero hay que comprobar la excentricidad de los esfuerzos verticales sobre la zapata. $e = \frac{\sum Momentos\ flectores}{\sum Axiles} = 0.78\ m$

Con ello sabemos que : $e > a/6$, y verificamos con la ley triangular

Realizando los pasos necesarios, llegamos a que:

$\sigma_1 \leq 1.25 \cdot \sigma_{adm} \rightarrow 1.31 \leq 1.25 \cdot 2.2 \rightarrow 1.31 \leq 2.75 \frac{kg}{cm^2}$, quedando verificado.

8.3.6. Cálculo de la armadura principal

Según el artículo 58.2.1 de la EHE-08, la distribución de tensiones no es lineal. Para su análisis utilizamos el Método de bielas y tirantes. Calculando la excentricidad sin incluir el peso propio e la zapata, y según el punto 5.8.4.1 de la EHE-08, obtenemos el área total de la armadura pasiva:

$$A_s = 16.40\ cm^2$$

Tras esto, chequeamos la mínima cuantía geométrica, según la tabla 42.3.5 de la EHE-08. El resultado final es que hay que poner una malla de $15x15\ cm$ hecha de barras de hacer de $12\ mm$ de diámetro.

8.3.7. Armadura del cáliz

Ficha técnica en el proyecto completo.

8.4. Cálculo de las vigas de atado

Su principal función será soportar los cerramientos, ya que serán pesados y podrían hundirse en el terreno.

8.4.1. Pre-dimensionado de las vigas

Del mismo modo que con las zapatas, pre-dimensionaremos y comprobaremos si satisface las necesidades:

$a_{beam} = 0.35\ m$; $b_{beam} = 0.35\ m$

8.4.2.Acciones a considerar para el cálculo de la viga.
Ficha técnica de los cerramientos en proyecto complete.
Tendré en cuenta el peso propio de los cerramientos y de las vigas de atado.

8.4.3.Comprobación de la armadura de la viga de atado
Ficha técnica en el proyecto completo.

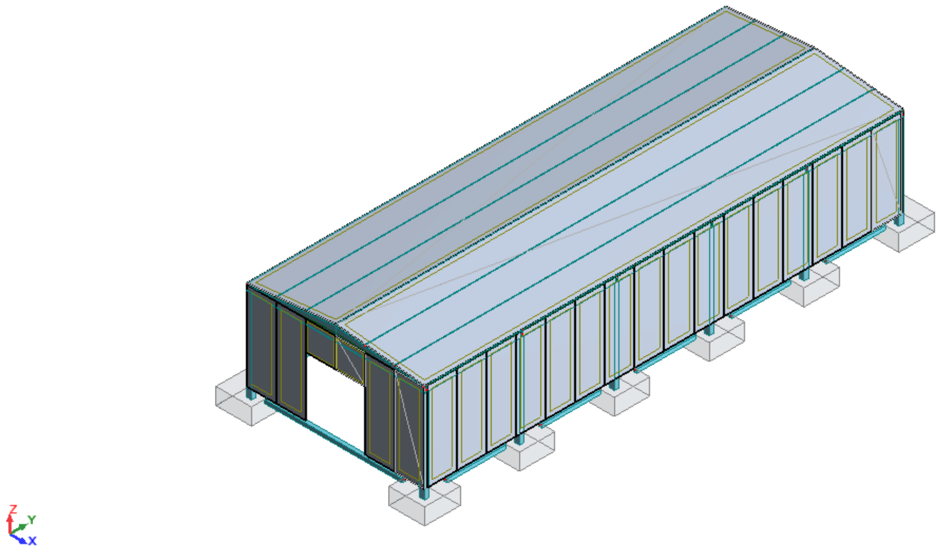
8.4.3.1. Verificación del cortante aplicado (ULS)" $\mathrm{V_d}$"
Para la verificación del cortante aplicado utilizaremos el punto 44.2.3.1 de la EHE-08: $V_d \leqslant V_{u1}\ y\ V_d \leqslant V_{u2}$. En este caso:
$V_d = 85.77 \leq 236 \leq 350\ kN$, quedando verificado.

8.4.3.2. Verificación del momento flector para fisura debido al viento (ULS) "$\mathrm{M_f}$"
De acuerdo a la tabla 8.2.2 de la EHE-08, la viga esta está sujeta a una exposición tipo IIa. La apertura máxima en este caso es $W_{max} = 0.3\ mm$. Dado que $W_k \leq W_{max} \rightarrow 0.2 \leq 0.3\ mm$, se cumple que $M_f \leq M_a \leq M_{f0.3}\ kN \cdot m$, por lo que la viga aguanta correctamente.

8.4.3.3. Verificación de la cuantía geométrica mínima de la armadura principal
La verificación de la cuantía geométrica mínima se hace de mismo modo que para la zapata. Su resultado es que es válido para este caso.

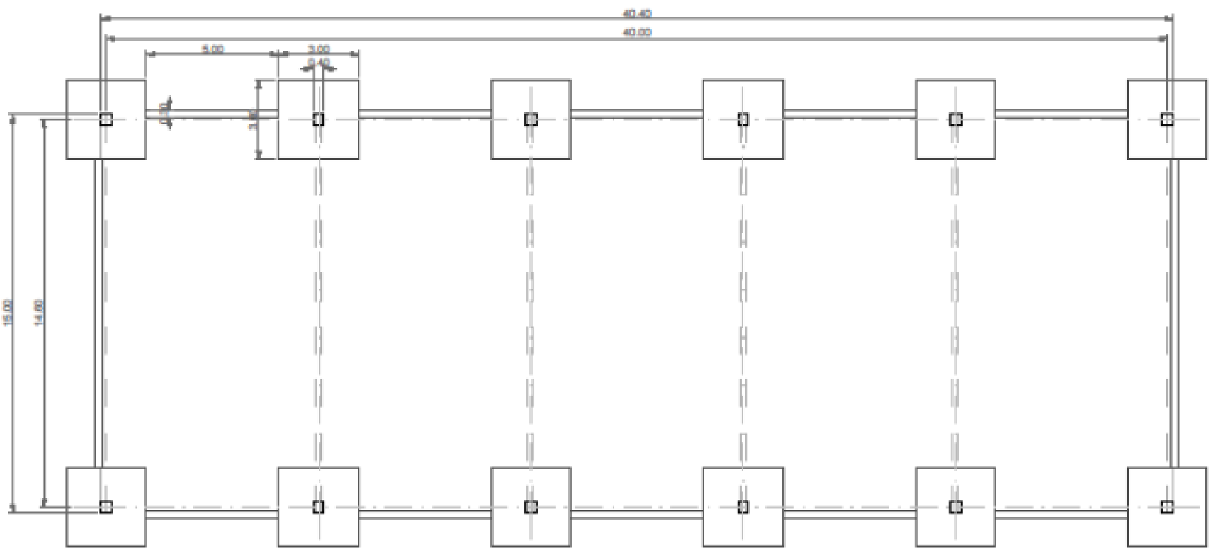
8.5. Disposición final de las zapatas y vigas de atado
Imágenes en el proyecto completo.

9. VISTAS DEL DISEÑO EN 3D



Resto de imágenes en el proyecto completo.

10. PLANOS ACOTADOS



Resto de imágenes en proyecto completo.

11. CONCLUSIONES

De acuerdo a los cálculos realizados, una nave industrial con dichas dimensiones soportaría las acciones aplicadas y sería estructuralmente segura.
Dado que los paneles de la entrada serán más ligeros que el resto de cerramientos, no ha sido necesario realizar distinciones a la hora de realizar cálculos sobre los elementos afectados por los mismos.
Como se puede comprobar, las vigas de atado no están completamente centradas, ya que su misión principal es soportar los cerramientos y que no se hundan en el terreno.

12. BIBLIOGRAFÍA

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13. PROGRAMA USADOS

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